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PERFORMANCE AND UPGRADING OF WASTEWATER STABILIZATION PONDS

Proceedings of a Conference
Held August 23-25, 1978, at Utah State University
Logan, Utah

Edited by

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. The complexity of the environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

As part of these activities, these proceedings were prepared to make available to the sanitary engineering community the results of many studies conducted by EPA and other agencies on the upgrading of wastewater lagoon effluents for the removal of algae, bacteria, and chemical components from lagoon effluent.

Francis T. Mayo
Director
Municipal Environmental Research
Laboratory

ABSTRACT

A conference, jointly funded by Utah State University and the Office of Research and Development, U.S. Environmental Protection Agency, on the performance and upgrading of wastewater stabilization ponds was held August 23-25, 1978, at the College of Engineering, Utah State University, Logan, Utah. The Proceedings contain 18 papers discussing and describing the design, operation, performance and upgrading of lagoon systems. Performance data for facultative and aerated lagoons collected at numerous sites throughout the USA are presented. Design criteria and the applicability of performance data to design equations are discussed. Rock filters, intermittent sand filters, microscreening and other physical-chemical techniques, phase isolation, land application, and controlled environment aquaculture were evaluated as methods applicable to upgrading lagoon effluents. The proceedings conclude with a presentation on the costs associated with the construction, operation and maintenance of lagoon systems.

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INTRODUCTORY REMARKS

Francis T. Mayo*

It is always a pleasure to share with peers the important fruits of research and developmental activities. This symposium on the performance and upgrading of wastewater stabilization ponds offers such an occasion. It is an important follow-up to the 1974 meeting (also held in Logan, Utah), devoted to summarizing preliminary results of pilot methods for removal of algae from lagoon effluents and other information on design and operation of wastewater lagoons. The results of EPA's recently concluded 5-year program on lagoon performance and upgrading will be summarized at this symposium along with important work by other organizations. The work includes data on performance of full-scale facultative and aerated lagoons; disinfection of lagoon effluents; and the use of slow-rock filters, intermittent slow-sand filters, phase-isolation ponds, and land application as means of upgrading lagoon effluents.

Lagoon systems have historically enjoyed some important advantages over more conventional wastewater treatment systems because of minimal power consumption, generally lower capital costs and lower operation and maintenance costs. However, the apparent simplicity of lagoon systems should not imply that haphazard designs or inadequate data can be tolerated, and still provide required levels of wastewater treatment. Because of the greater number of unknowns, lagoon systems can be more complex to design than conventional treatment systems.

The studies that are going to be reviewed indicate that well designed and operated wastewater lagoon systems can be tailored to meet various levels of effluent requirements and still retain the features attractive to small communities--low capital costs and low operating and maintenance costs.

It is important that we make available to consulting engineers, local governments, and regulatory agencies, sufficient knowledge of the design and performance of lagoon systems. They can then adequately consider the array of alternative systems available for solving individual wastewater treatment problems, especially for the smaller communities.

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INTRODUCTION AND OBJECTIVES OF SYMPOSIUM

Robert L. Bunch*

On behalf of the Municipal Environmental Research Laboratory of the U.S. Environmental Protection Agency (EPA), I wish to welcome you to this "Symposium on Performance and Upgrading Wastewater Stabilization Ponds." You are a select triad consisting of regulatory officials, researchers and implementors of stabilization ponds. In the next two and one-half days we intend to pass on to you the most recent results of our on-going research programs on stabilization ponds. Ample time also has been allotted for panel discussions. Hopefully, each participant has brought a problem he can present during the discussions. To find the best solutions to these problems will require your active participation in the group discussions.

This is the second time the Office of Research and Development of the U.S. Environmental Protection Agency and Utah State University have cooperated in presenting a symposium on wastewater stabilization ponds. The first meeting was during August 1974. The tone of the first meeting was quite different than the one here today. In 1974 there was an overall negative attitude as to the suitability of lagoons in meeting the secondary treatment requirements without supplemental treatment. There were several events that lead to that feeling. The Federal Water Pollution Control Act Amendments of 1972 has established the minimum performance requirements for publicly-owned treatment works. One of the requirements of this Act was that by July 1977 publicly-owned treatment works must meet effluent limitations based on secondary treatment as defined by the EPA administrator. On August 17, 1973, EPA published Secondary Treatment Standards in the Federal Register, Vol. 38, No. 159, Part II, p. 22298-22299. These regulations stated: (a) the five-day biochemical oxygen demand (BOD_5) and suspended solids (SS) shall not exceed an arithmetic mean value of 30 mg/l for effluent samples collected in a period of 30 consecutive days nor 45 mg/l for samples collected in seven consecutive days; (b) the arithmetic mean of the effluent BOD_5 and SS values determined on samples collected in a period of 30 consecutive days shall not exceed 15 percent of the arithmetic mean of BOD_5 and SS values determined on influent sampes collected at approximately the same times during the same period; and (c) the geometric mean of the fecal coliform bacteria in the effluent shall not exceed 200 per 100 milliliters for samples collected in a period of 30 consecutive days, nor 400 per 100 milliliters for samples collected in a period of seven consecutive days. A report "Lagoon Performance and the State of Lagoon Technology" prepared by George Barsom of Ryckman, Edgerly, Tomlinson and Associates, Inc., gave no assurance that there were existing lagoon designs capable of meeting the secondary treatment standards. As a result of these events, EPA decided to put research funds into developing inexpensive methods for upgrading lagoons. The first projects funded were to have a quick return so that the technology could impact the construction grants program and meet the July 1, 1977, deadline for achieving secondary treatment. Some of the upgrading techniques investigated were the intermittent sand filter, submerged rock filter and land application of algae laden effluents. The results of these projects were presented at the first symposium in 1974.

As a result of an extensive review of the literature on waste stabilization ponds and the information presented at the last lagoon symposium, EPA arrived at the following conclusions concerning lagoons. Of the 4,000 publicly-owned waste treatment lagoons for the United States, over 90 percent of them are located in small rural communities and are designed for a flow of less than 1 mgd. There is a wide variation in the design of the lagoon system and long-term performance data are generally lacking. Most of the results were from infrequent grab sampling, particularly for continuous discharge facultative lagoons and aerated lagoons. The data that were available indicated that multiple cell lagoons were better than one large lagoon and that the effluent quality is deteriorated either by large amounts of algae in lagoon effluents during the summer period or by icing over of the lagoons in winter, resulting in anaerobic conditions or simply by rising levels of soluble organics in the effluent due to the drastic decrease in biological activity during cold winters.

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*Robert L. Bunch is Chief, Treatment Process Development Branch, Wastewater Research Division, Municipal Environmental Research Laboratory, Environmental Protection Agency, Cincinnati, Ohio.

In the past 5 years, EPA has attempted to determine how effectively well-designed and well-managed waste treatment lagoons operate throughout all seasons of the year. Because of the preponderance of continuous-discharge facultative and aerated lagoons in municipal use, the evaluation was confined to these two types. Four facultative and five aerated wastewater treatment lagoon systems were included in our survey. The sites were carefully chosen so that ranges of climatic and regional conditions would be covered. In that the majority of the existing lagoons serve small communities, candidate lagoons serving around 5,000 people or less were chosen. The sampling and analytical data gathering period was for at least 1 year.

Since the last symposium, EPA has also sponsored grants on nutrient control, disinfection, and algae removal. The total outlay of research funds by EPA since 1973 on lagoon studies has been over $1.5 million.

Today there is a different attitude toward lagoons than at our meeting in 1974. Today, there is more confidence both in the capabilities of conventional pond systems and in the use of supplementary devices to upgrade the existing lagoons. The Environmental Protection Agency believes that wastewater treatment ponds play a vital role in the Nation's water pollution control strategy. Because of their advantages of simplicity, low cost and minimal energy requirements, ponds should always be considered as viable options of wastewater treatment for smaller communities. Two pieces of legislation passed by EPA that have aided the use of lagoons are deletion of the fecal coliform bacteria limitation from the definition of secondary treatment on July 26, 1976, and the granting of a variance on October 7, 1977, which allows the EPA Regional Administrator to adjust the suspended solids limitation for individual small ponds. This variance from the definition of secondary treatment applies only to ponds with a design capacity of 2 mgd or less. This standard applies to publicly owned treatment works and is not directly applicable to private or federal wastewater treatment ponds.

The Municipal Environmental Research Laboratory has initiated the preparation of a lagoon design manual. The manual will include design criteria, case histories, and cost estimates for all feasible types of stabilization ponds. It is estimated that it will take 2 years to prepare and publish this manual. This symposium is designed to get the latest information on lagoons into your hands today so that it will impact newly designed lagoons and give you guidance on upgrading of existing lagoons.

EPA desires to be responsive to your needs and will be guided by your recommendations for future studies on lagoons. If you have suggestions, please make them known to me here at the meeting or when you return home.

HISTORICAL REVIEW OF OXIDATION PONDS AS THEY IMPACT SECONDARY TREATMENT AND WATER QUALITY

Ronald F. Lewis*

BACKGROUND

The Federal Water Pollution Control Act Amendments of 1972 (the Act) established an extensive program to restore and maintain the biological, physical, and chemical quality of the Nation's waters. As part of this program, effluent limitations were to be established for point discharge waste sources. For municipal wastewater treatment the Act required EPA to define the effluent quality that can be achieved by secondary treatment. More stringent effluent requirements may be necessary to meet water quality standards.

For both municipal and industrial wastewater treatment, oxidation ponds can play a significant role in helping to meet the established effluent limitations. Oxidation ponds are relatively easy to design, construct, and maintain, hence their popularity for wastewater treatment. The basic simplicity of oxidation ponds results in a lack of control over the microbial and chemical reactions in the ponds. It is common to find that ponds are not able to achieve the required level of treatment when used as the only treatment process.

In spite of the fact that oxidation ponds have been used for wastewater treatment for decades, continuing research needs to be undertaken to improve oxidation pond performance to meet progressively stringent effluent limitations. In the mid-twenties, cities in California, Texas, and North Dakota were using lagoons as a means of treating municipal sewage. However, it was not until several years later that design guidelines were developed. Consequently, it was more by accident, the availability of a "pot-hole" perhaps, than by design that lagoons (oxidation ponds) evolved as a viable method for treating raw sewage.

DEFINITIONS

There are many types of aerobic wastewater treatment ponds which are used to achieve the objective of satisfactory wastewater treatment such as:

-photosynthetic ponds - These ponds are designed to rely on photosynthetic oxygenation for a portion of the oxygen needed for waste treatment. They primarily are facultative. Terminology used to describe these ponds includes: oxidation ponds, waste stabilization ponds or lagoons, aerobic lagoons, facultative lagoons.

-retention ponds - This type relies on evaporation and percolation to exceed inflow so that there may be no discharge during part or all of a year. Photosynthetic biological activity will exist in such ponds.

-aerated ponds - Such ponds do not rely solely on photosynthetic oxygenation; aeration is supplied by mechanical means. These ponds also contain algae (suspended solids generally 150 mg/l or less) unless, as in the case of the oxidation ditch, the detention period is so short and the aeration and mixing are sufficient to allow a high concentration of mixed liquor volatile suspended solids (generally 800 mg/l or more) inhibiting sunlight penetration and algal growth.

An oxidation pond is a large, relatively shallow basin designed for long term detention of a wastewater which may or may not have received prior treatment. While in the pond, biological activity oxidizes the influent wastes and results in the synthesis of microorganisms and algae. The effluent will contain low concentrations of soluble BOD and varying amounts of suspended solids in the form of microbial floc and algae.

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Oxidation ponds are widely used in the United States. From 45 lagoons in 1945 located in a few states, today the use has spread to all of the 50 states and over 4000 lagoons. Approximately 93 percent of the lagoons are 0.5 mgd or less according to recent statistics kept in the Storet File. This technique of treating municipal sewage has provided low cost treatment for many communities.

Ponds have been used because operation is simple, operating costs are low, and land is available. There is a wide variation in the design of the ponds. Pond loadings increase and retention times decrease from north to south in the United States, reflecting the effect of differing climatic conditions and their effect on performance.

Until the last four or five years, comprehensive performance data from oxidation ponds usually have not been available. Effluent sampling commonly is lacking or is infrequent grab sampling. Thus, there was a dearth of data that could relate effluent quality to design or actual loading, to variations in influent load, to climatic conditions, or to effluent limitations.

SECONDARY TREATMENT REQUIREMENTS

Publication of the definition of secondary treatment (1) in the early 1970s focused attention on the fact that oxidation ponds had difficulty in meeting the requirements. The effluent limitations for municipal wastewater treatment were:

> -The arithmetic mean of values of BOD_5 and suspended solids for effluent samples collected in a period of 30 consecutive days shall not exceed 30 mg/l respectively, and for effluent samples collected in a period of seven consecutive days shall not exceed 45 mg/l respectively.
>
> -The arithmetic mean of the BOD_5 and suspended solids for effluent samples collected in a period of 30 consecutive days shall not exceed 15 percent of the arithmetic mean of the values of the influent samples collected at approximately the same times during the same period (85 percent removal).
>
> -The geometric mean of the value for fecal coliform bacteria for effluent samples collected in a period of 30 consecutive days shall not exceed 200 per 100 milliliters and for effluent samples collected in a period of seven consecutive days shall not exceed 400 per 100 milliliters.
>
> -The effluent values of pH shall remain within the limits of 6.0 and 9.0.

An interpretation of 304 (d) of the 1972 FWPCF amendments was that the definition of secondary treatment should be based on technical considerations rather than water quality or economic considerations. In the record of legislative history of the 1972 Act, Congress seemed to indicate that the term "seconday treatment" be utilized in its broadest context, not limited to the definition to suspended solids and BOD. They recognized the fact that traditional methods of what was then called secondary treatment produced a range of effluent quality of 50-90 percent reduction (2).

LAGOONS AND SECONDARY TREATMENT

For years the subject of lagoons has been a topic for discussion. Numerous authors have cited a gamut of research needs. The first major meeting addressing the subject of lagoons (oxidation ponds, etc.) was held in 1960. E. E. Smallhorst of the Texas Department of Health called for more knowledge of ponds. He cited the problem of the algae green color in effluents as being the most pressing need followed by the need to determine the sanitary quality of pond effluents, and a definition of the limiting factor for control of algae laden effluents in a receiving body of water (3). Referring to ponds in the upper Midwest States, primarily North Dakota, J. Svore and W. Van-Heuvelen praised the merits of lagoons, but did not classify ponds as conventional secondary treatment (3). In the State of North Dakota today, all treatment of sewage is performed in lagoons with the exception of one community. However, there is a concern for effluent quality since most of the systems in that state are controlled discharge systems. In a controlled discharge system, discharge is allowed only a few times a year. The final cell is taken out of service and is allowed to remain in a state of quiescence for a period of time. The algae settle, then the operator discharges a very high quality effluent. This technique is practiced in other northern states.

Culp stated that whenever ponds were used for secondary treatment, it was an exception to the general practice of following conventional primary facilities with conventional secondary units. D. P. Green of Wyoming's Department of Public Health did go on record that pond effluents produced an effluent equivalent to the conventionally accepted methods of secondary treatment (3).

Ponds were the topic of a second international symposium in 1970. Middleton and Bunch cited high concentrations of suspended solids in effluents as one of the major disadvantages (4). They were concerned about the inability to predict the quality of effluent from ponds as confidently as one might predict the effluent quality of more conventional technology.

G. M. Barsom and D. W. Ryckman analyzed pond performance with respect to the requirements of the Water Quality Act of 1965. Barsom noted that:

> It is true that lagoons more closely approach natural purification than any other treatment process; however, what has not been fully recognized is that the objectives of natural purification and waste treatment are different. As discussed above, the goal of natural purification is to recycle pollutants. The goal of waste treatment is to remove pollutants. Oxidation ponds, which emulate natural purification, recycle rather than remove pollutants. This is the fundamental reason why lagoons are unable to achieve acceptable effluent quality and why lagoons should not be considered equivalent to secondary treatment. (5)

The author went on to note that in addition to quality failures, there were aesthetic failures by lagoons which could be grouped into the following: (1) obnoxious hydrogen sulfide odors, (2) malodorous algae blooms, (3) noxious vegetative growth, (4) mosquito breeding, (5) highly colored effluents, and (6) septic sewage odors. He found that in most cases studied, oxidation ponds did not significantly enhance water quality based upon reduction of suspended solids, volatile solids, chemical oxygen demand, nitrogen, and phosphorus.

In a later publication by Barsom, he noted that "suggestions that chlorination of effluent and the subsequent killings of algae cells does not degrade stream quality is unsupported." (6) He encouraged additional research on this subject as well as on algae removal systems. The report did include data on suspended solids which indicated that ponds would have great difficulty in meeting the requirements of secondary treatment as defined under the FWPCA amendments of 1972. Barsom noted that secondary treatment had traditionally been equated to biological treatment of wastewater (bacteria and microorganisms breakdown complex organic pollutants to more stable substances), and that "lagoons had been considered secondary treatment." Prior to the Act of 1972, there was no standard definition (in terms of pollutant concentrations) of secondary treatment although environmental engineers used the term to denote removal of dissolved organic materials as well as suspended solids.

EFFECTS OF ALGAE ON RECEIVING STREAMS

While many authors reported on the performance of ponds and attempted to relate that to design criteria, few included suspended solids removal efficiency data. In addition to Middleton and Bunch, D. L. King, R. C. Bain, and others cautioned against the probable impact of suspended solids (namely, algae) on receiving streams. King et al. (7) reported that algae-laden effluents can significantly influence the condition of the receiving stream; in fact, the receiving stream may become a part of the total waste treatment system. Bain et al. (8) reported that the oxygen demand loading from the pond they investigated exceeded that of the river upstream of the plant. These reports and a report for the EPA by Hill and Shindala (9) show that the BOD_5 value of an algal-laden lagoon effluent is often only 20-30 percent of the BOD_{26} and BOD_{30} values recorded. However, in many instances the rate of die-off and decay of the algae in the receiving stream has been slow enough to blend in with the transformations of other naturally occurring plant substances (leaves from trees, aquatic plants, shore plants) and does not create a nuisance condition in the receiving stream. Thus in many cases, there is no steep increase in oxygen demand below the lagoon effluent nor are there obnoxious odors. This fact has been recognized by the EPA in amending the suspended solids regulation concerning the effluent limitations for small publicly owned lagoon treatment systems.

REVISION OF SECONDARY TREATMENT STANDARDS

The definition of secondary treatment for federal regulation of municipal wastewater treatment plant effluents has been modified because of a continuing dialog between the EPA and state water pollution control agencies and researchers. This will be covered in more detail in the next paper of this symposium. The Federal Register, Vol. 41, No. 144, Monday, July 26, 1976, contains amendments pertaining to effluent values for pH and deletion of fecal coliform bacteria limitations from the definition of secondary treatment. The amendment allowing less stringent suspended solids limitations for wastewater treatment ponds is found in the Federal Register, Vol. 42, No. 195, Friday, October 7, 1977. The relaxation of the suspended solids limitations was based partly on evidence that in many instances release of live algae into receiving waters did not appear to create odors or adversely affect the receiving stream. It was also partly based on the decision that costs of adding algae removal devices or replacement of lagoon systems with other types of wastewater treatment plants would be too costly and take too much time to meet the 1977 deadline. States may still impose more stringent suspended solids limitations on treatment plants discharging to water quality limited streams.

DISINFECTION OF LAGOON EFFLUENTS

The removal of enteric bacteria and pathogens in wastewater lagoon systems has been well documented in the literature. Okun (10) reported 99.99 percent reduction of coliforms in each of five experimental ponds loaded at up to 100 pounds of BOD per acre per day. Geldreich et al. (11) in a study of raw sewage and effluent from a waste stabilization pond located at a state prison dairy farm, reported a reduction in coliform bacterial density from a low of 85.9 percent in the winter to a high of 94.4 percent in the autumn. Fecal coliform reductions were greater than 87.9 percent, while reductions of fecal streptococci were 97.4 percent or more. Coetzee and Fourie (12) reported a 99.98 percent reduction of E. coli in wastewater lagoons while Drews (13) found that E. coli was reduced 99.6 percent in summer and 96.8 percent in the winter. Amin and Ganapati (14) reviewed some of the theories proposed for the bacterial reduction such as the production of materials toxic to the bacteria, germicidal effects of sunlight, sedimentation, and competition for nutrients. Using a laboratory-scaled lagoon, they also correlated the reduction of the enteric bacteria with biochemical changes occurring as the lagoon treatment of the wastewater progressed. The growth of the algae, Chlorella, and production of extra cellular fatty acids appeared to have an antibacterial effect. Gann et al. (15) found coliform reduction to be closely associated with BOD removal indicating that the coliforms may have been removed because of their inability to compete successfully for nutrients. Coetzee and Fourie (12) feel that because of the ease of elimination of E. coli in stabilization ponds that it may not be an infallible indicator of pathogenic organisms in wastewater lagoon effluents. Malchow-Moller et al. (16) considered that Streptococcus faecalis and Clostridum welchii were more reliable indicators of pathogens in such lagoon effluents.

Although fewer tests for pathogens have been conducted in lagoon system influents and effluents than tests for the indicator organisms, most results show a die-away of the pathogens. Coetzee and Fourie (12) found the total reduction of Salmonella typhi in the effluent from two stabilization ponds operated in series with a detention time of 20 days to be 99.5 percent at a time when the reduction of E. coli was 99.98 percent. Using Staplococuus aureus and Serratia marcesens as test organisms, Conely et al. (17) found an almost complete die-off of these organisms in the upper aerobic zone of a lagoon in 36 to 48 hours. Christie (18) found that field lagoons reduced the titre levels of polio virus from 10^5 to less than 10^3 units per milliliter.

Little, Carroll, and Gentry (19) and Slanetz et al. (20) indicate that series operation of lagoon systems produce better bacterial quality effluents than do single-celled systems. Slanetz (20) showed that when the ponds were operated in a series of three or four ponds, very few viable indicator bacteria remained in the effluents from the last lagoon in the series when the temperature of the lagoons ranged from 10 to 26°C. Counts as low as 2 per milliliter or less were obtained for coliforms, fecal coliforms, and fecal streptococci. Salmonella could be detected in only 1 out of 24 effluent samples from the third or fourth lagoon in the series during the summer. However, enteric viruses were isolated from a number of these effluent samples. The percentage survival of coliforms, fecal coliforms, and fecal streptococci was much higher during the winter when the temperature in the lagoons ranged from 1 to 10°C. The isolation of enteric pathogens was also more frequent during the winter from the lagoon effluents. Joshi, Parhad, and Rao (21) tested the performance of two lagoon systems located at Nagpur, India, with respect to the reduction of *Salmonella*, coliforms, *E. coli*, and fecal streptococci. *Salmonella* were not completely eliminated from the effluent of a two-celled lagoon that had submerged connections between the cells, while none were found in 1 liter samples of effluent from the third cell of a three-celled lagoon with the cells connected by a surface overflow arrangement. The authors concluded that the number of cells and interconnecting arrangements are important design features to be considered as well as loading rates and detention times for lagoon systems. Using a six-celled lagoon system near Oklahoma City, Oklahoma, Carpenter et al. (22) showed that no pathogens were found in the wastewater beyond the first two lagoon cells in series or in any of the 179 fish sampled from those grown in cells three to six.

Marais (23) presents a consolidated theory for a kinetic model for the reduction of fecal bacteria in stabilization ponds incorporating the effect of temperature on the specific death rate as well as discussing effects of mixing, anaerobic conditions, recycling of effluent, and the relationship between a single pond, a series of ponds, and batch conditions. The rate constant K (die-off rate) is very sensitive to temperature and is approximately related to $K = 2.6\ (1.19)^{T-20}$, (T°C). It is presumed in this relationship that the ponds are mixed and aerobic or facultative and is valid between 5 to 21°C. Above 21°C, with low wind velocities, periods of stratification occur causing the lower liquid depth of the pond to be anaerobic. There is a decline in the K value under anaerobic conditions; the reduction of fecal organisms is sharply reduced. When winter temperatures are very low, the K is very small and series operation will yield only relatively minor improvement over single pond operation.

Sobsey and Cooper (24) found that in algal-bacterial treatment systems both virus adsorption to solids and virus inactivation due to microbial activity play a role in reducing the enteric virus concentration in wastewater. In laboratory cultures the growth of the alga *Scedesmus quadricauda* and the bacterium *Bacillus megaterium* in sterile sewage had detrimental effect on polio virus survival, whereas the growth of heterogenous populations of stabilization pond bacteria in the same medium resulted in substantial virus inactivation.

Horn (25) showed that selective chlorination can be achieved for lagoon effluents under controlled conditions, whereby coliforms are destroyed, yet leaving the algae essentially intact. Control of time of reaction and concentration of chlorine is essential, however, excessive chlorine can release materials from algal cells and increase the effluent BOD.

Melmed (26) reported that gamma radiation or radiation plus chlorine could be used for the disinfection of an effluent from a maturation pond.

HEAVY METALS AND ALGAE

The question has been raised as to whether or not the algae growing in wastewater lagoons can carry significant amounts of harmful heavy metals through the treatment process and into the effluent to cause subsequent problems in the receiving stream. Like other plants, algae can adsorb and absorb many minerals; they have a requirement for small concentrations of many metals needed for their enzyme systems. O'Kelly (27) gives an excellent review of the inorganic nutrients of algae covering 16 elements. He includes information concerning accumulation of some elements by certain algae and information concerning toxicity effects towards algae. There are different effects at the same concentration of metal for different algae and the effects are conditioned by other factors in the growth and nutrition of the algae.

It is often uncertain, as Ramani (28) pointed out in reporting on the removal of chromium in the series operated Napa, California, lagoon system, as to whether the metal was adsorbed, assimulated, or precipitated to the bottom of the lagoon cells. Reed et al. (29) show that no excessive concentrations of heavy metals occur in forage grasses receiving lagoon effluents at sites where the effluents have been used for irrigation for periods ranging from 10 to 76 years.

Andrew et al. (30) showed that a fresh water alga, *Chara braunii*, concentrated radioactive manganese-54 to approximately six times the level found in vertebrates and invertebrates in the same water.

Emery et al. (31) reported on the accumulation of Pu transuranics in the algae of a waste pond which has been receiving Pu processing wastes for approximately 30 years. Hart and Scaife (32) reported the bioaccumulation of cadmium in Chlorella pyrenoidosa. The organism had the ability to accumulate large concentrations of Cd before showing adverse effects, such as decreased CO_2 fixation or decreased O_2 evolution. Wixson (33) showed how the accumulation of metals by algae could be used in the purification of the effluent from Pb/Zn mining and smelting operations.

A number of the heavy metals, however, have an inhibitory or toxic effect on many of the algae. Greene et al. (34) and Miller et al. (35) describe the toxicity of zinc to the green alga Selenastrum capricornutum and also demonstrated that zinc levels in the Spokane River had an effect on limiting the amount of algal growth that occurred. Hassall (36) studied the effects of copper on Chlorella vulgaris and suggests that the toxic effect of the copper may be its effect on the phosphorus metabolism of the growing cells or that the copper renders the cell permeable to solutes. Copper has frequently been used as a fungicide or as an algicide.

TASTES AND ODORS FROM ALGAE

An excellent review of the different types of tastes and odors that are associated with algae in water is given by Palmer (37). Typical descriptions of some of the odors encountered are aromatic, fishy, grassy, musty, or earthy, and pig-pen or septic. Tables are presented by Palmer showing the type of odor and taste associated with a number of algal genera from a large variety of algal groups.

Jenkins et al. (38) identified by gas chromatographic techniques several odorous sulfur compounds including methyl mercaptan, dimethyl sulfide, isobutyl mercaptan, and n-butyl mercaptan in cultures of blue-green algae. Most of these compounds arose from bacterial putrefaction of the blue-green algal cells. However, the organism Microcystis flos-aquae was shown to produce isopropyl mercaptan during periods of active growth. Safferman et al. (39) found that Symploca muscorum, a blue-green alga, produced an earthy-smelling metabolite at an estimated concentration of 0.6 mg/l of culture medium. The substance was identified as Geosmin by a direct comparison to an actinomycete produced standard. Medsker et al. (40) proved that there was no actinomycete contamination of the S. muscorum culture obtained from R. S. Safferman. The compound Geosmin, $C_{12}H_{22}O$, was shown to be a dimethyl substituted, saturated, two-ring tert-alcohol. The growth of the blue-green alga Oscillatoria in fish ponds in Israel caused an off-flavor in carp from the fish pond (41). Lovell and Scakey (42) showed that channel catfish aquired the distinct earthy-musty flavor associated with Geosmin when grown in tanks containing dense masses of either Symploca muscorum or Oscillatoria tenuis. The fish acquired the flavor within 2 days; flavor intensity reached a maximum at 10 day.

Silvey et al. (43) discussed some of the control methods, such as forced aeration and recirculation in impoundments to prevent stratification and growth of the odor producing blue-green algae. Treatment with chlorine and use of activated carbon does help to minimize or remove Geosmin from the water. A bacterial oxidation of Geosmin is described by Narayan and Nunez (44) that may also be used to reduce the odor problem.

AQUATIC TOXICITY FROM ALGAE

Certain algae may produce substances that can elicit a variety of clinical responses from fish, animals, and man. The clinical manifestations may range from digestive disturbances (vomiting and diarrhea) to organ damage, neuromuscular disfunction, and death. The most common groups causing toxic problems are the blue-green algae in fresh waters and flagellates or chrysomonads in marine waters. The marine forms are the producers of an alkaloid that is concentrated in shellfish (it is nonpoisonous to the shellfish) and is the paralytic shellfish poison that has caused numerous human deaths. The blue-green algae have caused numerous fish kills and deaths of many types of animals. Schwimmer and Schwimmer (45) present a masterly review of the medical problems caused by algae. They reference over 250 individual research papers covering animal intoxications from algal blooms (blue-green algae), experimental testing of bloom material on test animals, and the toxic manifestations following the experimental administration of these naturally occurring toxic fresh water algae, toxic manifestations following experimental administration of laboratory cultured algae, effects on fish in fresh and salt waters, and effects on humans.

HEALTH EFFECTS OF ALGAE

The toxic effects of the blue-green algae have been studied extensively. Gorham (46,47) reports on laboratory studies on the toxins produced by blue-green algae. At least two toxins produced by strains of Microcystis aeurginosa and Anabena flos-aquae are responsible for acute poisonings of animals. Bacteria associated with the algae also produced toxins responsible for less acute poisonings. Microcystis fast-death factor is a cyclic polypeptide of moderate toxicity which kills livestock and other animals but not waterfowl. The structure of Anabena very-fast-death factor was not elucidated. It killed a variety of animals, including waterfowl. Maloney and Carnes (48) showed that the fast-death factor toxic from Microcystis while lethal for mice had no effect on three species of fish or Daphnia. Aziz (49) isolated the toxin in a non-dialyzable fraction of the lysate from whole cells of Microcystis and showed that this material caused diarrhea in the litigated small intestinal loops in guinea pigs. Carmichael et al. (50) showed that the main effect of a toxin isolated from bacteria-free lyophilized suspensions of Anabena flos-aquae was production of a substantial postsynaptic depolarizing neuromuscular blockage; the animals died as a result of respiratory arrest.

In the review by Schwimmer and Schwimmer (45), references are given to work by Dillenberg (51), Hayami and Shino (52,53,54), and McDowell (55,56) that show that green algae can also be causative agents of human gastrointestinal disorders besides those caused by blue-green algae. Bernstein and Safferman (57,58,59,60) in a series of papers showed that animals and man can have allergic responses to green algae. This was shown to be elicitated by skin sensitivity and by nasal challenge with aerosolized extracts of Chlorella vulgaris. The authors showed that viable green algae occurred in house dust. Of 37 patients exhibiting allergic responses to a 1:10,000 dilution of house dust, 22 showed positive reactions to one or more of the algal extracts diluted 1:10,000 and 5 to extracts diluted 1:1,000. Thus, house dust may be a likely source of human exposure and sensitization to many varieties of algae, which may give rise to clinical allergic problems.

EPA RESEARCH PROGRAM

Recent literature (61,62,63,64) continues to cite the problems of ponds with the control of algae as one of the major problems. The most frequently cited problems associated with oxidation ponds over the years had been odors and organic matter in the effluent. With the increased concern over the ability of oxidation ponds to meet the effluent limitations, concerns were more sharply focused on the solids, nutrients, and bacteria in the effluent. All of these problems and concerns can be related to the drainage, maintenance, and control of the biological reactions of the ponds.

The general research needs related to the use of oxidation ponds as a wastewater treatment alternative are and continue to be:

-Design and operation as related to effluent quality
-Seasonal and climatic performance variations
-Cost effectiveness to delineate where supplemental aeration equipment or use of an aerated lagoon is preferable
-Control of excessive solids in the effluent
-The type of nutrient control that may be effective
-Disinfection needs and appropriate methodology
-Need for supplemental nutrients with industrial wastes
-Land disposal methodology for oxidation pond discharge
-Protection of groundwater quality

R&D PROGRAM OBJECTIVES

The Environmental Protection Agency embarked upon a research and development program designed to identify design criteria and technology to upgrade the effluent quality of oxidation ponds. The specific research objectives identified by EPA in conjunction with state and local officials were:

a. Low cost suspended solids removal processes

b. Design guidelines for discharging oxidation pond effluent to the land

c. Disinfection guidelines and demonstration of applicable disinfection methods

d. Development and demonstration of nutrient control technology

e. Evaluation of utilizing the separated suspended solids in the pond effluent as a useful product, such as a supplement for animal feed

f. Relationship of pond design to an upgrading of pond performance (performance evaluations of facultative and aerobic systems)

These needs are applicable to ponds used for the treatment of both municipal and industrial wastewaters.

R&D PROGRAM STATUS

A specific research and development program related to the performance of oxidation ponds was established. Several large scale projects were funded with preliminary results reported at other symposia and the full results will be presented in greater detail at this meeting.

Under design, construction and/or operation are intermittent sand-filters, submerged rock filters, chemical addition technology, phase-isolation systems, and systems applying effluent to the land. Results of these studies will also be presented at this symposium.

PRELIMINARY COSTS

The results from the studies to date have permitted an order of magnitude estimate of the costs of upgrading oxidation ponds. To upgrade all known existing oxidation ponds in the country would cost about $2 billion, whereas replacing all ponds with mechanical plants was estimated to cost about $5 billion. More specifically, the cost of add-on devices such as slow-sand intermittent and slow-rock filters, and chemical addition was estimated as about $1000/1000 gallons compared to about $2000-$2900/1000 gallons for conversion to a mechanical plant of the same capacity (about 0.2 mgd). Oxidation ponds can be upgraded by any one of several add-on devices or by employing more land to facilitate conversion to a controlled discharge system, whichever is the more cost-effective alternative. Based upon the recent research results, it appears that upgrading oxidation ponds is a viable alternative.

SUMMARY

We may summarize lagoon treatment systems without algal removal or efforts to minimize the amount of algae in the effluent as having the following effects:

1. Ultimate oxygen demand of the receiving water will be high if the algae accumulate in sludge banks or settle out in basins. Approximately 2/3 of the algal mass may decay. This could lead to a low DO in the receiving water.

2. Enteric indicators of fecal pollution and pathogens decrease in the treatment process. Differences in temperature, detention time, and short circuiting in some lagoon systems lead to all pathogens being removed without the necessity of the use of chlorine or other disinfectants in some cases and not in others. Algae in the effluents can interfere with the efficiency of the disinfection process.

3. Most small towns have little in the way of heavy metal contamination of the raw wastewater. In a few industrial lagoon systems described in the literature, the heavy metals precipitated in the anaerobic layers at the bottom of the lagoon cells.

4. Extensive masses of algae can lead to taste and odor problems. Release of algae from lagoon effluents and subsequent decay can release nutrients (nitrogen and phosphorus) that were removed from the wastewater by the growth of the algae in the wastewater lagoon cells. These nutrients may aid in the development of the obnoxious blue-green algae. Poor management of the wastewater lagoon systems may allow extensive growths of blue-greens directly in the lagoon system, creating odor problems.

5. Extensive growths of algae may cause allergies or other clinical problems to animals or men that come into close contact with the masses of algae by swimming in the receiving water, ingesting the water, or breathing an aerosol of these algae. This is especially a problem where there are large masses of decaying algae.

Methods have been developed to economically prevent algae from reaching the effluent or removing the algae from the effluent in a polishing system that can be used by small communities where operators are only infrequently at the waste treatment plants. With proper design, operation and algae removal processes, wastewater lagoon systems are one of the most reliable, least costly, and most effective waste treatment systems.

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RECENT AMENDMENTS TO SECONDARY TREATMENT REGULATIONS FOR SUSPENDED SOLIDS IN LAGOONS

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Amendments to the Secondary Treatment Regulations (40 CFR Part 133.103) which were promulgated by the Environmental Protection Agency on September 28, 1977, and which became effective on November 7, 1977, allow less stringent suspended solids limitations for wastewater treatment ponds for small municipalities. The amended regulations and the official notice of this action are contained in the October 7, 1977, Federal Register, pages 54664 through 54666. Modification of the suspended solids limitations is not a blanket action applicable to all wastewater treatment ponds. Instead, modifications are allowable on a case-by-case basis for pond systems in which waste stabilization ponds are the sole process used for secondary treatment, where the maximum facility design capacity is two million gallons per day or less and where operational data indicate that a monthly average suspended solids concentration of 30 mg/l cannot be achieved. In some states the amended regulations are already being implemented, in some states the revised suspended solids values which will be allowed are still under development, and in some states there are no plans to allow any variance from the suspended solids values specified by the secondary treatment regulations.

As most of you are aware, Public Law 92-500--the Federal Water Pollution Control Act--became law on October 18, 1972. This Act gave the Environmental Protection Agency 60 days to define secondary treatment, and it required all Publicly Owned Treatment Works (POTW's) to achieve the effluent quality obtainable with secondary treatment by July 1, 1977. You are also probably aware that many POTW's were unable to meet the July 1, 1977, deadline for secondary treatment. In some cases this was because federal financial assistance was not available and the needed facilities could not be constructed in time, and in other cases it was because the facilities that were built could not meet the specified effluent quality.

From the beginning there was much concern as to whether waste stabilization pond systems could meet the secondary requirements, particularly for suspended solids concentrations. However, the agency had missed its 60 day statutory deadline and it was under considerable pressure to promulgate the secondary treatment regulations when this action was finally accomplished and published in the Federal Register in August 1973. As you know, the initial secondary treatment regulations provided for meeting the following basic effluent limitations:

BOD_5 and Suspended Solids--
  30 mg/l (monthly average)
BOD_5 and Suspended Solids--
  45 mg/l (weekly average)
Fecal coliform--200 per 100 ml
  (30 day geometric mean)
Fecal coliform--400 per 100 ml
  (7 day geometric mean)
pH--6.0 to 9.0

Almost immediately after the regulations were adopted, the agency held several meetings to further evaluate the waste stabilization lagoon problem. The agency did not want to drive conventional wastewater treatment lagoon systems out-of-use as secondary treatment devices, yet, there was mixed opinion as to whether or not conventional two and three cell systems could meet secondary requirements on a consistent basis. In March 1974, it was decided to publish a Technical Bulletin on wastewater treatment ponds as additional guidance relative to the use of lagoon systems in the construction grants program.

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For flow-through photosynthetic ponds, the Bulletin indicated that grants would only be made where there was reasonable assurance of satisfactory performance. A determination of satisfactory performance could be based upon a similar pond system in a similar environment or the performance of a pilot project. In addition, the Bulletin called for the facility plans for pond systems to include discussions of actions to be taken if upgrading was determined to be necessary after the system was placed in operation.

In early 1975, EPA began discussing possible revisions of the secondary treatment regulations. On July 26, 1976, the first set of modifications of the secondary treatment regulations was adopted. However, at this time there was no adjustment of the suspended solids requirements. The first set of modifications removed the effluent limitations for fecal coliform organisms, and it provided for modifications of the effluent values for pH when due to photosynthetic activity not caused by inorganic chemicals added to the wastewater or chemicals from industrial sources. Subsequently on September 2, 1976, a second modification of the secondary treatment regulations was proposed which would allow relaxation of the suspended solids limitations for certain lagoon systems of small communities. As mentioned earlier, this second modification was adopted and became effective on November 7, 1977.

The latest change in the secondary treatment regulations reflects a desire on the part of the Environmental Protection Agency to allow waste stabilization pond systems to continue in use as viable secondary treatment processes without the need for extensive upgrading to meet suspended solids limitations. In short, the agency wants small communities to be able to continue to take advantage of the low construction cost and the less complex operation and maintenance that is possible with the use of pond systems as secondary treatment devices. Where higher degrees of treatment are needed to meet water quality standards or other considerations, ponds can also continue to be used effectively in many situations with various upgrading techniques that have been developed through recent research and demonstration projects.

What is the significance of the new regulations and what are the conditions that are attached to their use? First, it should be recognized that Section 510 of the Clean Water Act gives any state the right to require effluent limitations more stringent than federal requirements. Therefore, those states which do not choose to allow relaxed effluent limitations for suspended solids in lagoon systems have every right to maintain more strict requirements.

The regulations also apply only to Publicly Owned Treatment Works and not to industrial lagoon systems.

As indicated earlier, the relaxed effluent limitations are to be applied only on a case-by-case basis to pond systems which meet standards for good design but which cannot otherwise meet secondary treatment requirements for suspended solids. The relaxed standards apply to pond systems where ponds are the only means of secondary treatment and only to pond systems where the maximum facility design capacity is 2 mgd or less. While the secondary requirements for fecal coliform organisms have been deleted, and the pH requirements for ponds have been relaxed, the pond systems must meet the 30 mg/l (monthly average) and 45 mg/l (weekly average) values for BOD_5. In addition, they must meet relaxed standards for suspended solids except that relaxed standards for suspended solids would not be authorized in any case where such relaxation would result in a water quality standards violation in the receiving stream. In such cases, regular secondary treatment requirements would have to be met, or treatment higher than secondary would be required as necessary to meet the water quality standards.

What are the relaxed standards or effluent limitations that must be met? The regulations say that the treatment works must conform with the suspended solids concentration achievable with best waste stabilization pond technology. Best waste stabilization pond technology is, in turn, defined to mean "a suspended solids value, determined by the Regional Administrator (or, if appropriate, the State Director subject to EPA approval), which is equal to the effluent concentration achieved 90 percent of the time within a State or appropriate continguous geographical area by waste stabilization ponds that are achieving the levels of effluent quality established for biochemical oxygen demand in Section 133.102a" of the secondary treatment regulations. In other words the suspended solids value achieved 90 percent of the time is a statistical number for suspended solids for pond effluents that also meet the BOD_5 requirements.

These "90 percent of the time" suspended solids values have been established in a number of states, and they range from a low value of around 40 mg/l to high values slightly greater than 100 mg/l. In practice these numbers will be used in NPDES permits as 30 consecutive day average values or average values over the period of discharge when the entire duration of discharge is less than 30 days.

Within the next couple of months the Environmental Protection Agency intends to publish in the Federal Register a listing of the values that are being used in the various states as the suspended solids concentration to be achieved 90 percent of the time. This notice will also provide further explanation of the program. The numbers that have been developed to date are not considered permanent and will be re-evaluated as additional data are available. While it is not likely that any major changes will be made, some minor changes might be made in the future.

It is also probable that some states will be re-evaluating their design standards and design policies relative to waste stabilization ponds. These re-evaluations will be made with the idea that certain upgrading may be desirable in pond design to insure that newly designed pond systems conform to the ideas of best technology.

EVALUATION OF FACULTATIVE WASTE STABILIZATION POND DESIGN

Brad A. Finney and E. Joe Middlebrooks*

INTRODUCTION

The principal objectives of this paper are to outline the performance of existing lagoon systems and to evaluate several facultative waste stabilization pond design equations. To satisfy the need for reliable lagoon performance data, in 1974 the U.S. Environmental Protection Agency sponsored four intensive facultative lagoon performance studies. These studies were located at Peterborough, New Hampshire (EPA, 1977a), Kilmichael, Mississippi (EPA, 1977c), Eudora, Kansas (EPA, 1977d), and Corinne, Utah (EPA 1977b). These studies encompassed 12 full months of data collection, including four separate 30-consecutive-day sample periods once each season.

A number of equations have been proposed to design facultative wastewater stabilization lagoons. Design engineers must choose between these often contradictory methods when designing a facultative pond system that will provide adequate wastewater treatment at a reasonable cost. These design techniques include simple design criteria based on organic loading and hydraulic detention time, empirical design equations, and rational design equations. Examples of each technique will be used in conjunction with the data collected at the four sites described above.

SITE DESCRIPTION

Peterborough, New Hampshire

The Peterborough facultative waste stabilization lagoon system consists of three cells operated in series with a total surface area of 8.5 hectares (21 acres) followed by chlorination. A schematic drawing of the facility is shown in Figure 1. A chlorine residual of 2.0 mg/l is maintained at all times. The facility was designed in 1968 on an areal loading basis of 19.6 kg BOD_5/day/ha (17.5 lbs BOD_5/day/ac) with an initial average hydraulic flow of 1893 m^3/day (0.5 mgd). At the design depth of 1.2 m (4 ft), the theoretical hydraulic detention time would be 57 days. The results of the study conducted during 1974-1975 indicated an actual average areal loading of 15.6 kg BOD_5/day/ha (13.9 lbs BOD_5/day/ac) and an average hydraulic flow of 1011 m^3/day (0.267 mgd). Thus, the actual theoretical hydraulic detention time was 107 days.

Kilmichael, Mississippi

The Kilmichael facultative waste stabilization lagoon system consists of three cells operated in series with a total surface area of 3.3 ha (8.1 acres). The effluent is not chlorinated. A schematic drawing of the facility is shown in Figure 2.

The design load for the first cell in the series was 67.2 kg BOD_5/day/ha (60 lbs BOD_5/day/ac). The second cell was designed with a surface area equivalent to 40 percent of the surface area of the first cell. The third cell was designed with a surface area equivalent to 16 percent of the first cell. The system was designed for a hydraulic flow of 693 m^3/day (0.183 mgd). The average depth of the lagoons is approximately 2 m (6.6 ft). This provides for a theoretical hydraulic detention time of 79 days. The result of the study indicated that the actual average organic load on the first cell averaged 27.2 kg BOD_5/day/ha (24.3 lbs BOD_5/day/ac) and that the average hydraulic inflow to the system was 281 m^3/day (0.074 mgd). Thus, the actual theoretical hydraulic detention time in the system was 214 days.

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Eudora, Kansas

The Eudora facultative waste stabilization lagoon system consists of three cells operated in series with a total surface area of 7.8 ha (19.3 ac). A schematic diagram of the system is shown in Figure 3. The effluent is not chlorinated.

The facility was designed on an areal loading basis of 38 kg BOD_5/day/ha (34 lbs BOD_5/day/ac) with a hydraulic flow of 1514 m^3/day (0.4 mgd). At the designed operating depth of 1.5 m (5 ft), the theoretical hydraulic detention time would be 47 days. The results of the study indicated that the actual average organic load on the system was 19.0 kg BOD_5/day/ha (16.7 lbs BOD^5/day/ac) and the actual average hydraulic flow to the system was 506 m^3/day (0.13 mgd). Thus, the actual theoretical hydraulic detention time in the system was 231 days.

Corinne, Utah

The Corinne facultative waste stabilization lagoon system consists of seven cells operated in series with a total surface area of 3.86 ha (9.53 ac). A schematic drawing of the system is shown in Figure 4. The effluent is not chlorinated.

The facility was designed on an areal loading basis of 36.2 kg BOD_5/day/ha (32.2 lbs BOD_5/day/ac) with a hydraulic flow of 265 m^3/day (0.07 mgd). With a design depth of 1.2 m (4 ft), the system has a theoretical hydraulic detention time of 180 days. The results of the study indicated that the actual average organic load on the system was 14.1 kg BOD_5/day/ha (12.6 lbs BOD_5/day/ac) and the actual average hydraulic flow to the system was 694 m^3/day (0.18 mgd). Thus, the actual theoretical hydraulic detention time in the system was 70 days.

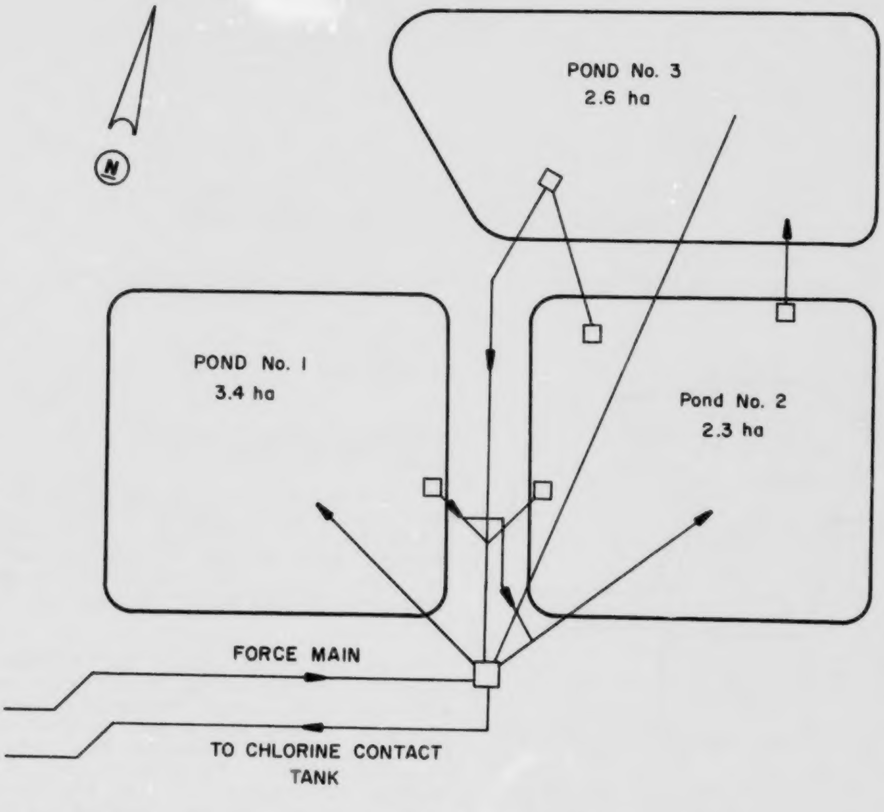

Figure 1. Facultative lagoon system at Peterborough, New Hampshire (EPA, 1977a).

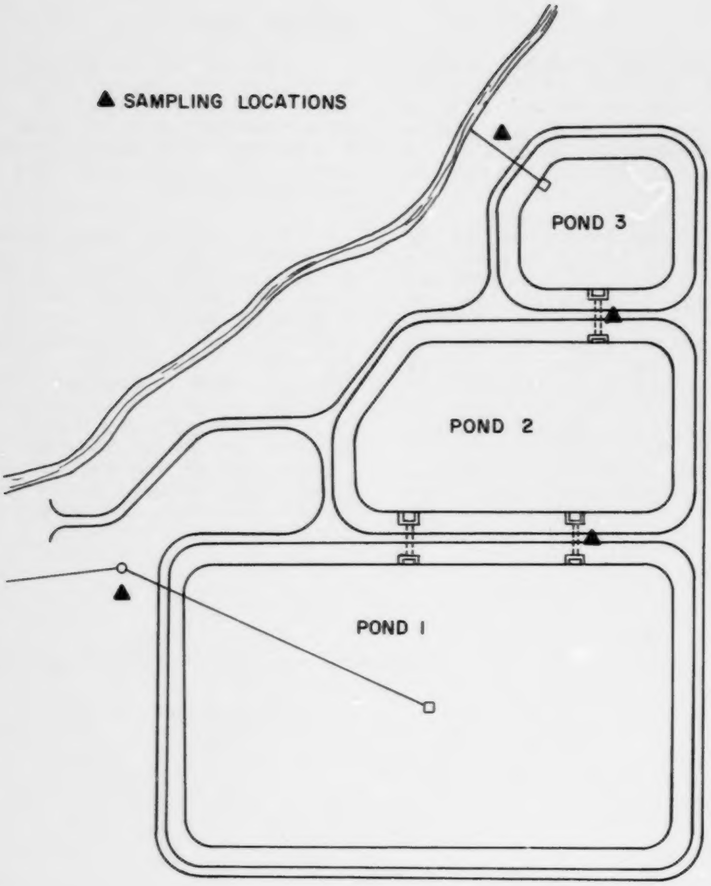
▲ SAMPLING LOCATIONS
POND 3
POND 2
POND I

Figure 2. Facultative lagoon system at Kilmichael, Mississippi (EPA, 1977c).

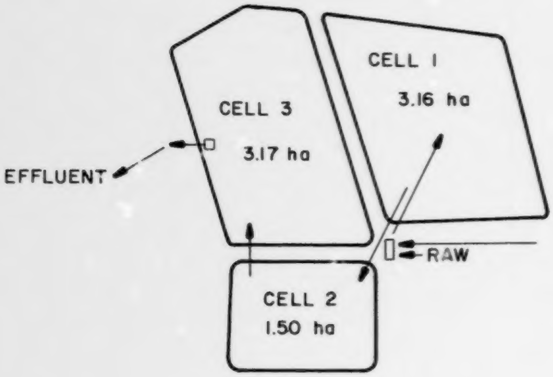
CELL I
3.16 ha
CELL 3
3.17 ha
EFFLUENT
RAW
CELL 2
1.50 ha

Figure 3. Facultative lagoon system at Eudora, Kansas (EPA, 1977d).

PERFORMANCE

Biochemical Oxygen Demand (BOD_5) Performance

The monthly average effluent biochemical oxygen demand (BOD_5) concentrations for the four previously described facultative lagoon systems are compared with the Federal Secondary Treatment Standard of 30 mg/l in Figure 5.

In general, all of the systems were capable of providing a monthly average effluent BOD_5 concentration of less than 30 mg/l during the major portion of the year. Monthly average effluent BOD_5 concentrations ranged from 1.4 mg/l during September 1975 at the Corinne, Utah, site, to 57 mg/l during March 1975

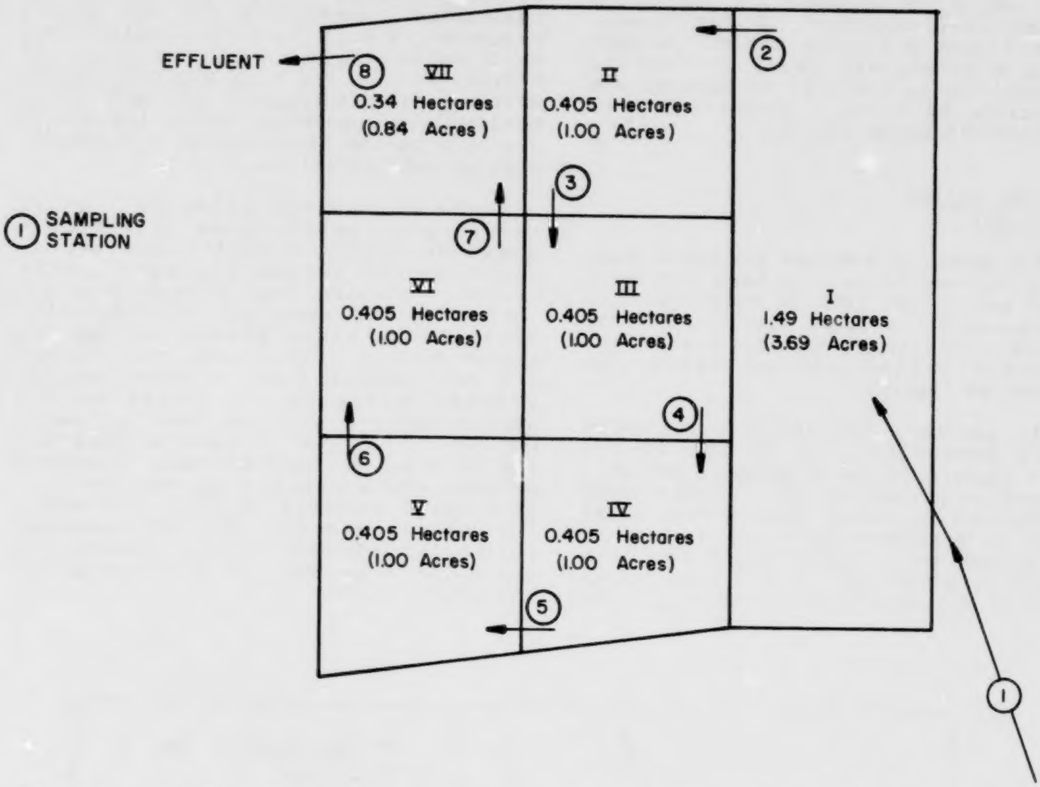

Figure 4. Facultative lagoon system at Corinne, Utah (EPA, 1977b).

at the Peterborough, New Hampshire, site. Monthly average effluent BOD_5 concentrations tended to be higher during the winter months (January, February, March, and April) at all of the sites. This was especially evident at the Peterborough site where the ponds were covered over by ice due to freezing winter temperatures. The ice cover caused the ponds to become anaerobic. However, even when the ponds at the Corinne site were covered over with ice the monthly average effluent BOD_5 concentration did not exceed 30 mg/l.

None of the systems studied were significantly affected by the fall overturn. However, the spring overturn did cause significant increases in effluent BOD_5 concentrations at two of the sites. At the Corinne site two different spring overturns occurred. The first occurred in March 1975, with a peak daily BOD_5 concentration of 36 mg/l. The second occurred during April 1975, with a peak daily effluent BOD_5 concentration of 39 mg/l. At the Eudora site, the peak daily effluent BOD_5 concentration of 57 mg/l occurred during April 1975. The Kilmichael and Peterborough sites were not severely affected by the spring overturn period.

The monthly average effluent BOD_5 concentration of the Corinne lagoon system never exceeded 30 mg/l throughout the entire study. The Eudora lagoon system monthly average effluent BOD_5 concentration exceeded 30 mg/l twice during the entire study. The Kilmichael lagoon system monthly average effluent BOD_5 concentration exceeded 30 mg/l on only two occasions during the study. The Federal Secondary Treatment of 30 mg/l was exceeded by the Peterborough lagoon system monthly average effluent BOD_5 concentration 4 of the 12 months studied.

The results of these studies indicate that properly designed, maintained, and operated facultative waste stabilization pond systems can produce a high quality effluent. Although these systems are subject to seasonal upsets, they are capable of producing a low biochemical oxygen demand (BOD_5) effluent which is suitable for polishing by various processes. Since facultative lagoon effluents exceed 30 mg/l during a relatively small portion of the year, it is possible to control the discharge in such a manner as not to exceed discharge standards.

Suspended Solids Performance

The monthly average effluent suspended solids concentrations for each system are illustrated in Figure 6. At the present there is no specific Federal Secondary Treatment Standard effluent suspended solids concentration for facultative lagoons.

In general, the effluent suspended solids concentrations of the facultative lagoons follow a seasonal pattern. Effluent suspended solids concentrations are high during summer months when algal growth is intensive and also during the spring and fall overturn periods when settled suspended solids are resuspended from bottom sediments due to mixing. The monthly average suspended solids concentrations ranged from 2.5 mg/l during September 1975 at the Corinne site to 179 mg/l during April 1975, also at the Corinne site. The high monthly average effluent concentration of 179 mg/l at the Corinne site occurred during the spring overturn period which caused a resuspension of settled solids.

The Eudora and Kilmichael sites illustrate the increase in effluent suspended solids concentrations due to algal growth during the warm summer months. However, the Peterborough and Corinne sites were not significantly affected by algal growth during the summer months. In general, the Corinne and Peterborough sites produced monthly average effluent suspended solids concentrations of less than 20 mg/l. During 10 of the 13 months studied, the monthly average effluent suspended solids concentration at the Corinne site never exceeded 20 mg/l. However, the monthly average effluent suspended solids concentration at the Eudora site was never less than 39 mg/l throughout the entire study.

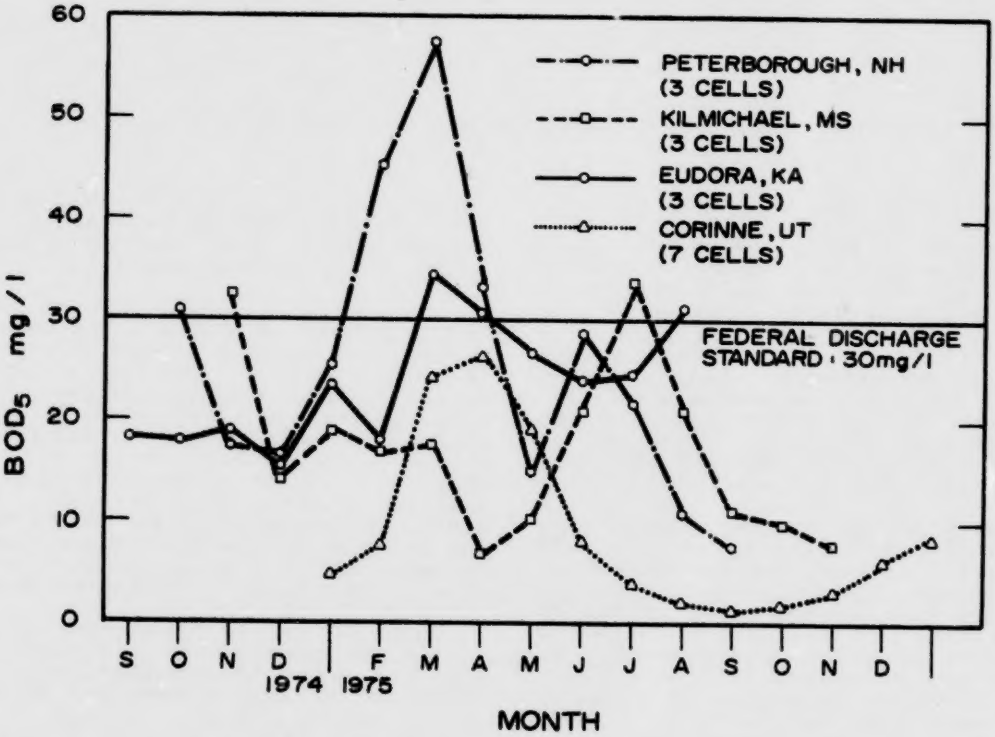

Figure 5. Monthly average effluent biochemical oxygen demand (BOD_5).

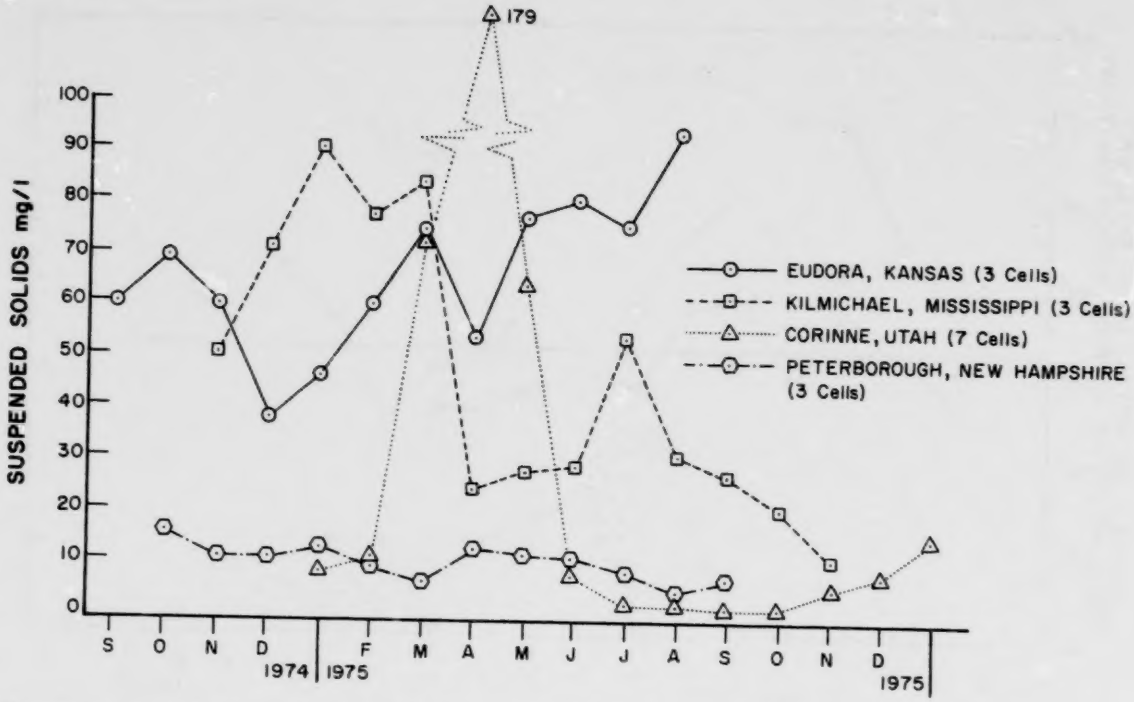

Figure 6. Monthly average effluent suspended solids for typical facultative lagoons.

The results of the studies indicate that facultative lagoons can produce an effluent which has a low suspended solids concentration. However, effluent suspended solids concentrations will be high at various times throughout the year. In general, these suspended solids are composed of algal cells which may not be particularly harmful to receiving streams. In areas where effluent suspended solids standards are stringent, some type of polishing device will be necessary to reduce facultative lagoon effluent suspended solids concentrations to acceptable levels.

Fecal Coliform Removal Performance

The monthly geometric mean effluent coliform concentrations for the four facultative lagoon systems are compared with a concentration of 200 per 100 ml in Figure 7.

Only the Peterborough, New Hampshire, facultative lagoon system employs chlorination. The other three facilities do not practice disinfection. As illustrated in Figure 7, the chlorinated Peterborough lagoon effluent never exceeds a concentration of 10 fecal coliform organisms per 100 ml. This clearly indicates that facultative lagoon effluent may be satisfactorily disinfected by the chlorination process.

For the three facultative lagoon systems without disinfection processes, the geometric mean monthly effluent fecal coliform concentration ranged from 0.1 organisms/100 ml in June and September 1975, at the Corinne, Utah, lagoon system to 13,527 organisms/100 ml in January 1975, at the Kilmichael, Mississippi, lagoon system. In general, geometric mean effluent fecal coliform concentrations tend to be higher during these periods. Periods of ice cover during winter months would seriously affect fecal coliform die-off due to sunlight effects. The Eudora, Kansas, and the Kilmichael, Mississippi, geometric mean monthly effluent fecal coliform concentrations consistently exceeded 200 organisms/100 ml during winter operation.

The Corinne, Utah, lagoon system never exceeded 200 organisms/100 ml

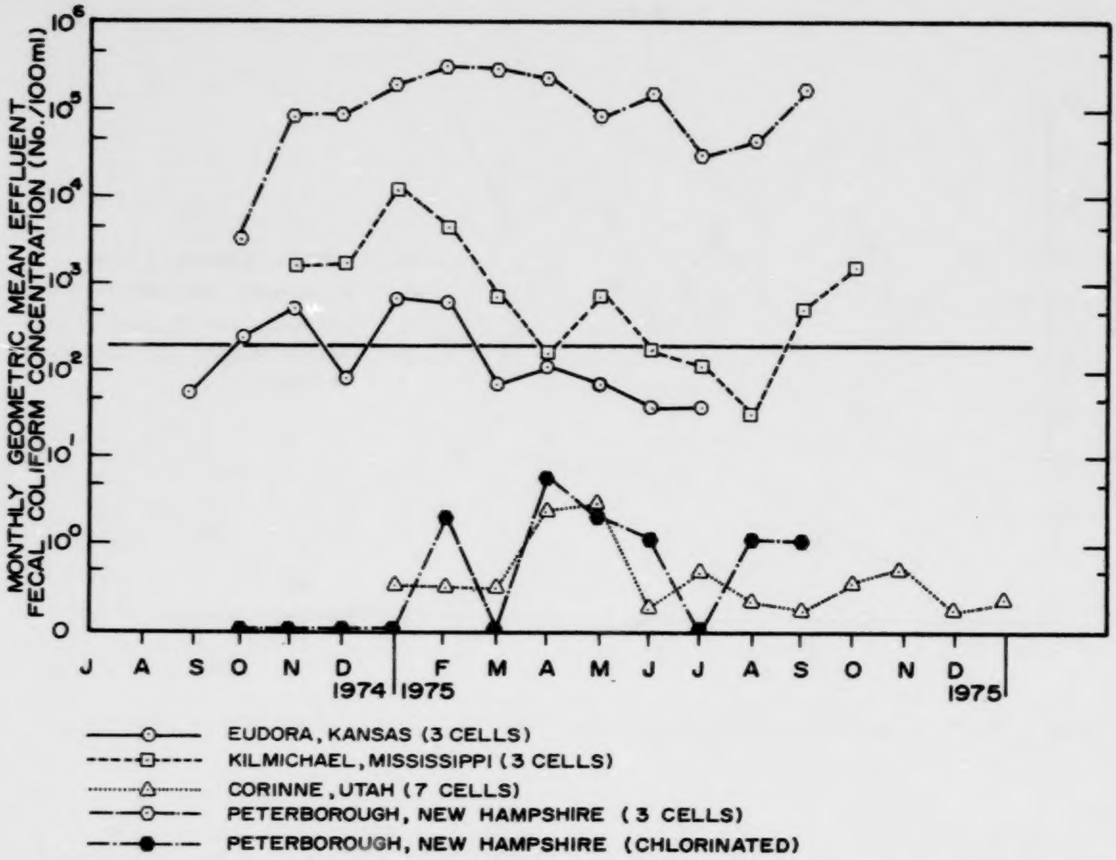

Figure 7. Effluent monthly geometric average fecal coliform concentrations from typical facultative lagoons.

even though this system did not practice any form of disinfection. This system is composed of seven cells in series. Analysis of the fecal coliform concentrations between the seven cells indicated that fecal coliforms were essentially removed after the fourth cell in the series (EPA, 1976b). The other two facultative lagoon systems without disinfection only utilize three cells in series. However, fecal coliform die-off is primarily a function of hydraulic residence time rather than the absolute number of cells in series.

The results of these studies indicate that facultative lagoon effluent can be chlorinated sufficiently to produce fecal coliform concentration less than 10 organisms per 100 ml. Two of the systems studied could not produce an effluent containing less than 200 fecal coliform/100 ml. This was probably due to hydraulic short circuiting. However, the Corinne, Utah, system study clearly indicated that properly designed facultative lagoon systems can significantly reduce fecal coliform concentrations.

EVALUATION OF DESIGN METHODS

Design Criteria for Organic Loading and Hydraulic Detention Time

Canter and Englande (1970) reported that most states have design criteria for organic loading and/or hydraulic detention time for facultative waste stabilization ponds. Design criteria are used by the states to ensure that pond effluent water quality meets state and federal discharge standards. Repeated violations of effluent quality standards by pond systems that meet state design criteria indicate the inadequacy of the criteria. Reported organic loading design criteria averaged 21.2 kg BOD_5/ha/day (26.0 lb BOD_5/ac/day) in the north region (above 42° latitude), 49.4 kg BOD_5/ha/day (44 lb BOD_5/ac/day) in the southern region (below 37° latitude) and 37.0 kg BOD_5/ha/day (33 lb BOD_5/ac/day) in the central region. Reported design criteria for detention time averaged 117 days in the north, 82 days in the central, and 31 days in the south region.

Design criteria for organic loading in New Hampshire is 39.3 kg BOD_5/ha/day (35 lb BOD_5/ac/day). The Peterborough treatment system was designed for a loading of 19.6 kg BOD_5/ha/day (17.5 lb BOD_5/ac/day) in 1968 to be increased as population increased to 39.3 kg BOD_5/ha/day (35 lb BOD_5/ac/day) in the year 2000. Actual loading during 1974-1975 averaged 16.2 kg BOD_5/ha/day (14.4 lb BOD_5/ac/day) with the highest loading being 21.2 kg BOD_5/ha/day (18.9 lb BOD_5/ac/day). Although the organic loading was substantially below the state design limit, the effluent exceeded the federal standard of 30 mg BOD_5/l during the months of October 1974, February, March, and April 1975.

Mississippi's design criteria for organic loading is 56.2 kg BOD_5/ha/day (50 lb BOD_5/ac/day). The Kilmichael treatment system was designed for a loading of 43 kg BOD_5/ha/day (38 lb BOD_5/ac/day). Actual loading during 1974-1975 averaged 17.5 kg BOD_5/ha/day (15.6 lb BOD_5/ac/day) with a maximum of 24.7 kg BOD_5/ha/day (22 lb BOD_5/ac/day) and yet the federal BOD_5 effluent standard was exceeded twice during the sample year (November and July).

The design load for the Eudora, Kansas, system was the same as the state design limit, 38.1 kg BOD_5/ha/day (34 lb BOD_5/ac/day). Actual loading during 1974-1975 averaged only 18.8 kg BOD_5/ha/day (16.7 lb BOD_5/ac/day) with maximum of 31.5 kg BOD_5/ha/day (28 lb BOD_5/ac/day). The federal BOD_5 effluent standard was exceeded 3 months during the sample year (March, April, and August).

Utah has both an organic loading design limit, 45 kg BOD_5/ha/day (40 lb BOD_5/ac/day) on the primary cell and a winter detention time design criteria of 180 days. Design loading for the Corinne system was 36.2 kg BOD_5/ha/day (32.2 lb BOD_5/ac/day) and design detention time was 180 days. Although the organic loading averaged 33.6 kg BOD_5/ha/day (29.8 lb BOD_5/ac/day) on the primary cell, during two months of the sample year it exceeded 56.2 kg BOD_5/ha/day (50 lb BOD_5/ac/day). Average organic loading on the total system was 13.0 kg BOD_5/ha/day (14.6 lb BOD_5/ac/day), and the hydraulic detention time was estimated to be 88 days during the winter. Regardless of the deviations from the state design criteria, the monthly BOD_5 average never exceeded the federal effluent standard.

A summary of the state design criteria for each location and actual design values for organic loading and hydraulic detention time are shown in Table 1. Also included is a list of the months the federal effluent standard for BOD_5 was exceeded. Note that the actual organic loading for all four systems are nearly equal, yet as the monthly effluent BOD_5 averages shown in Figure 5 indicate, the Corinne system consistently produces a higher quality effluent. This may be a function of the larger number of cells in the Corinne system; seven as compared to three for the rest of the systems. Hydraulic short circuiting may be occurring in the three cell systems resulting in a shorter actual detention time than exists in the Corinne system. Detention time may also be affected by the location of pond cell inlet and outlet structures. As shown in Figure 4, the outlet structures are at the furthest point possible from the inlet structures in the Corinne, Utah, system. At the Eudora, Kansas, system shown in Figure 3, large "dead zones" undoubtedly occur in each cell due to the unnecessarily short distance between inlet and outlet structures. These dead zones result in decreased hydraulic detention and increased effective organic loading rate. The extremely poor performance of the Peterborough system during the winter months suggests a need for New Hampshire to evaluate their organic loading standards.

Empirical Design Equations

In a survey of primary facultative ponds in tropical and temperate zones, McGarry and Pescod (1970) found that areal BOD_5 removal (L_r, lb/ac/day) may be estimated through knowledge of areal BOD_5 loading (L_o, lb/ac/day) using Equation 1.

$$L_r = 9.23 + 0.725\, L_o \quad . \quad . \quad . \quad . \quad (1)$$

It was reported that the regression equation had a correlation coefficient of 0.995 and a 95 percent confidence interval of $\pm$ 29.3 lb BOD_5/ac/day removal. The equation was reported to be valid for any loading between 30 and 500 lb BOD_5/ac/day. McGarry and Pescod also found that under normal operating ranges, hydraulic detention time and pond depth have little influence on percentage or areal BOD_5 removal.

Equation 1 was applied to the primary cell of the Peterborough, Eudora, and Corinne facultative pond systems. The Kilmichael system could not be used because the primary cell loading was below 30 lb/ac/day. The equation predicted higher removal (L_r) than was observed for all three systems although the observed data fell within the 95 percent confidence interval. The results obtained from Equation 1 are summarized in Table 2 and a plot of the equation and observed data is shown in Figure 8. As can be seen from Figure 8, the relative magnitude of the 95 percent confidence interval at loadings commonly found in the United States (under 100 lb/BOD_5/ac/day) makes the equation practically useless as a tool for facultative wastewater stabilization pond design. For example, at a loading of 80 lb BOD_5/ac/day, Equation 1 predicts, at a 95 percent confidence level, a range of removals from 37.9 to 96.5 lb BOD_5/ac/day.

Larsen (1974) proposed an empirical design equation, developed by using data from a one-year study at the Inhalation Toxicology Research Institute, Kirtland Air Force Base, New Mexico. The Institute's facultative pond system consists of one 0.66 ha (1.62 ac) cell receiving waste from 151 staff members, 1300 beagle dogs, and several thousand small animals.

Larsen found that pond surface area could be estimated by use of the following equation.

$$MOT = (2.468^{RED}+2.468^{TTC}+23.9/TEMPR+150.0/DRY)*10^{6} \quad . \; . \; . \; . \; . \; (2)$$

Table 1. Summary of state design standards, design and actual values for organic loading and hydraulic detention time.

Location	Organic Loading ($kg\ BOD_5/ha/day$) State Design Standard	Design	Actual (1974-1975)	Theoretical Hydraulic Detention Time (Days) State Design	Standard Design	Actual	Months Effluent Exceeded 30 mg/l BOD_5
Peterborough, NH	39.3	19.6	16.2	None	57	107	Oct., Feb., Mar., Apr.
Kilmichael, MS	56.2	43.0	17.5	None	79	214	Nov., July
Eudora, KS	38.1	38.1	18.8	None	47	231	Mar., Apr., Aug.
Corinne, UT	45.0*	36.2*	29.7* 14.6**	180	180	70 88***	None

(kg/ha/day) * 0.889 = (lb/ac/day)

*Primary Cell
**Entire System
***Estimated From Dye Study

Table 2. Actual and predicted areal BOD removal from primary facultative pond cells.

Location	Primary Cell Average Annual Areal BOD Loading (Lo) (lb/ac day)	Primary Cell Average Annual Areal BOD Removal (lb/ac day)	McGarry and Pescod Equation Areal BOD Removal (Lr) (lb/ac day)	McGarry and Pescod Equation 95% Confidence Interval (lb/ac day)
Peterborough, NH	36.5	24.2	35.7	$6.39 \leq L_r \leq 65.0$
Eudora, KS	42.0	35.0	39.7	$10.4 \leq L_r \leq 69.0$
Corinne, UT	32.9	19.2	33.1	$3.78 \leq L_r \leq 62.4$

where the dimensionless products

$$MOT = \frac{\text{surface area (solar radiation)}^{1/3}}{\text{influent flow rate (influent BOD}_5)^{1/3}}$$

$$RED = \frac{\text{influent BOD}_5 - \text{effluent BOD}_5}{\text{influent BOD}}$$

$$TTC = \frac{\text{wind speed (influent BOD}_5)^{1/3}}{\text{(solar radiation)}^{1/3}}$$

$$TEMPR = \frac{\text{lagoon liquid temperature}}{\text{air temperature}}$$

DRY = relative humidity

Table 3 lists the units for the parameters needed in Equation 2. Table 4 lists the dimensionless products, and the unit conversion factors required to make them dimensionless when calculated in the units indicated in Table 3. According to Larsen, since the design

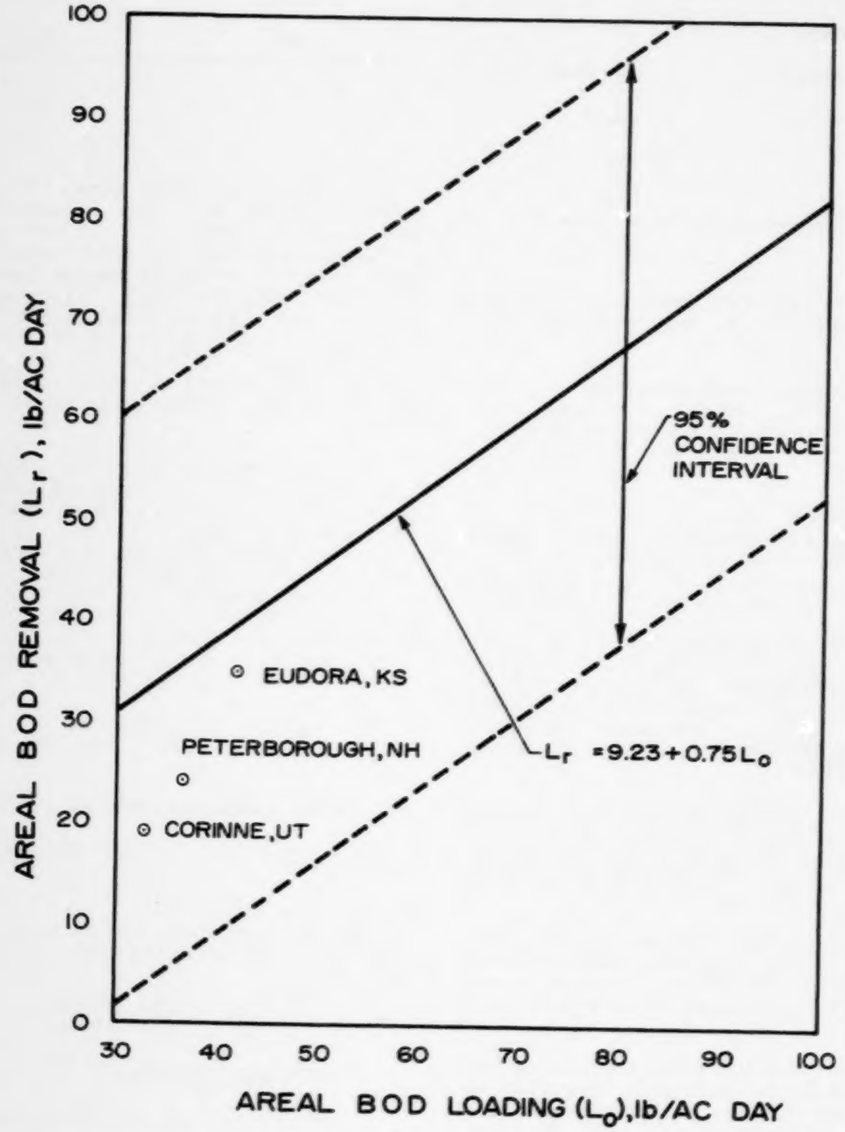

Figure 8. McGarry and Pescod equation for areal BOD_5 removal as a function of BOD_5 loading.

Table 3. Larsen equation input parameter units.

Parameter	Units
Wind Speed	Miles/Hour
Solar Radiation	BTU/Ft^2 Day
Relative Humidity	%
Air Temperature	°F
Lagoon Temperature	°F
Influent Flow Rate	gal/day
Influent BOD	mg/l
Effluent BOD	mg/l
Pond Area	Ft^2

Table 4. Dimensionless products and unit conversion factors.

Dimensionless Product	Unit Coversion Factor
MOT	1.0783 x 107
RED	1
TTC	0.0879
TEMPR	1.0
DRY	1.0

is in dimensionless form, it may be correctly applied to any geographical area.

To calculate pond area required for a specific BOD_5 removal, Larsen suggests using the least favorable climatic conditions. Pond depth must be from 3 to 5 feet.

To ensure that a maximum pond surface area was obtained when applying the Larsen equation to the four facultative waste stabilization pond systems, the following rules were observed.

1. Use winter (average of December and January) solar radiation data. With the exception of the Corinne system, solar radiation data were obtained from Visher (1966). The solar radiation data for the Corinne system were obtained from EPA (1977b).

2. Use lowest monthly relative humidity and yearly average windspeed. These data were obtained from the nearest reporting weather station listed by NOAA (1974).

3. Use winter (average of December and January) BOD_5 influent and effluent concentration, hydraulic loading, pond temperature, and daily average air temperature.

To determine the effect of using the Larsen equation on multicell facultative ponds, it was applied to both the entire system and the primary cell for each location. The data used in Larsen's equation verification are shown in Table 5.

The actual and predicted pond surface areas from the Larsen equation are shown in Table 6. In each case the Larsen equation underestimated the pond surface area required for a particular BOD_5 removal. The equation proved totally useless in designing multiple cell lagoons with prediction errors ranging from 190 to 248 percent. Prediction errors for the single cell surface areas range from 18 to 98 percent.

Predicted pond area was found to be insensitive to the BOD_5 removal fraction (the dimensionless product RED). For example, the Corinne, Utah, site predicted pond area was 1.12 ha for 100 percent BOD_5 removal (RED=1) while the predicted area was 1.06 ha for zero percent BOD_5 removal (RED=0). Predicted pond area was also found to be insensitive to the dimensionless product TTC. Therefore, letting

$$2.468^{RED} + 2.468^{TTC} = \text{constant} = C$$

The Larsen equation reduces to

$$\text{MOT} = (C + 23.9/\text{TEMPR} + 150/\text{day})*10^6 \quad \ldots\ldots (3)$$

Substituting for the dimensionless products and solving for surface area (A) yields:

$$A = \left\{\left[C + 23.9\left(\frac{\text{air temp.}}{\text{pond temp.}}\right) + \frac{150}{\text{relative humidity}} * 10^6\right]\right\} \times \left\{\frac{(\text{influent flow rate})(\text{influent BOD}_5)}{(\text{solar radiation})^{1/3}}\right\}$$

Table 5. Input data used for Larsen equation.

Location	Wind Speed (MPH)	Solar Radiation (BTU/Ft^2 Day)	Relative Humidity (%)	Pond Temperature (°F)	Air Temperature (°F)	Influent flow Rate (gal/day)	Influent BOD_5 (mg/l)	System Effluent BOD_5 (mg/l)	Primary Cell Effluent BOD_5 (mg/l)
Peterborough, NH	5.8	460	48	37.8	23.8	$2.36x10^5$	133.	11.6	45.0
Kilmichael, MS	7.6	735	47	48.2	44.6	$7.43x10^4$*	139.	23.1	22.5
Eudora, KS	10.9	827	46	46.2	44.3	$1.17x10^5$	363.	17.1	45.2
Corinne, UT	2.0	606	23	35.0	27.7	$7.66x10^4$	120.	7.2	21.0

Table 6. Predicted and actual pond surface area.

Location	Actual Surface Area Primary (ha)	Actual Surface Area Total (ha)	Predicted Surface Area Primary (ha)	Predicted Surface Area Total (ha)	Prediction Error For Primary (%)	Prediction Error For Total (%)
Peterborough, NH	3.40	8.50	2.87	2.93	18	190
Kilmichael, MS	2.10	3.21	1.06	1.06	98	203
Eudora, KS	3.16	7.82	2.32	2.33	36	236
Corinne, UT	1.49	3.86	1.10	1.11	34	248

(ha) * 0.405 = ac

Surface area is no longer a function of pond performance but of organic loading, hydraulic loading and climatic condition; therefore, the Larsen equation has less merit than state design criteria based on organic loading and hydraulic detention time.

Another empirical equation for the design of facultative waste stabilization ponds was proposed by Gloyna (1976). The equation, useful in determining pond surface area, is as follows:

$$V = 3.5 \times 10^{-5} QL_a \left[\theta^{(35-T)}\right] ff' \quad . . (4)$$

where

V = pond volume (m^3)
Q = influent flow rate (l/day)
L_a = ultimate influent BOD_u or COD (mg/l)
θ = temperature coefficient
T = pond temperature (°C)
f = algal toxicity factor
f' = sulfide oxygen demand

The BOD_5 removal efficiency can be expected to be 80 to 90 percent based on unfiltered influent samples and filtered effluent samples. A pond depth of 1.5 m is suggested for systems with significant seasonal variations in temperature and major fluctuations in daily flow. Surface area design using the Gloyna equation should always be based on a 1 m depth. According to Gloyna, the algal toxicity factor (f) can be assumed to be equal to 1.0 for domestic wastes and many industrial wastes. The sulfide oxygen demand (f) is also equal to 1.0 for SO_4 equivalent ion concentration of less than 500 mg/l. Gloyna also suggests the use of the average temperature of the pond in the critical or coldest month. Sunlight is not considered to be critical in pond design but may be incorporated into the Gloyna equation by multiplying the pond volume by the ratio of sunlight in the particular area to the average found in the southwest U.S.

The data used to evaluate the Gloyna equation is shown in Table 7. Since ultimate BOD data were not available, COD data were used. To smooth out wild fluctuations in influent flow rate and influent COD, averages from the 3 months with the lowest pond temperatures were used. The coldest monthly average cell temperature was used for the pond temperature. Both the algal toxicity factor and sulfide oxygen demand factor were assumed equal to 1.0. The solar radiation data are the January monthly average as reported by Visher (1966). The solar radiation in the southwest United States was assumed to be 300 cal/cm^2 day (Visher, 1966). The depth of the Peterborough and Corinne systems at 1.2 m is less than recommended by Gloyna. The depth of the Kilmichael and Eudora systems are 2 m and 1.5 m respectively.

The actual pond area, BOD$_5$ removal efficiency, and suggested pond area as determined by the Gloyna equation without the sunlight correction are presented in Table 8. The areas determined by the equation are substantially larger than the actual area for most ponds in the Kilmichael, Eudora, and Corinne systems yet the actual BOD$_5$ removal falls within the range expected (80 to 90 percent). Pond areas determined by the Gloyna equation are somewhat smaller than the actual areas for the Peterborough system, yet BOD$_5$ removal is considerably less than expected. Since the solar radiation at all the sites is less than the solar radiation in the southwest U.S., use of the sunlight correction would only improve the equation predictions for the Peterborough system. The inconsistent results obtained from the Gloyna equation point out its weakness as a design tool.

Rational Design Equations

Marais (1970) proposed three kinetic models of increasing complexity describing facultative waste stabilization pond performance. Only the first two models will be discussed here.

Model 1 assumes: 1) Complete and instantaneous mixing of influent with the pond contents, hence effluent BOD$_5$ equals pond BOD$_5$; 2) Degradation is

Table 7. Influent flow rate, influent COD, pond temperature and solar radiation data used in Gloyna equation.

Location	Influent Flow Rate, Q (l/Day)	Influent COD, La (mg/l)	Pond Temperature, T (°C)	Solar Radiation (cal/cm^2/day)
Peterborough, NH	2.69×10^5			125
Pond 1		222	2.7	
Pond 2		154	2.0	
Pond 3		136	2.0	
Kilmichael, MS	2.81×10^5*			225
Pond 1		256	8.9	
Pond 2		124	8.4	
Pond 3		117	8.0	
Eudora, KS	5.25×10^5			250
Pond 1		606	2.9	
Pond 2		172	3.2	
Pond 3		133	3.1	
Corinne, UT	4.87×10^5			180
Pond 1		168	1.2	
Pond 2		110	0.8	
Pond 3		98	0.8	
Pond 4		105	0.9	
Pond 5		100	0.8	
Pond 6		87	1.1	
Pond 7		79	1.4	

(l/day) * 0.264 = gal/day

*Monthly data not available, annual average

Table 8. Comparison of actual pond area and BOD_5 removal efficiencies compared to areas determined by Gloyna equation without sunlight correction.

Location	Actual Pond Area (ha)	BOD_5 Removal Efficiency* (%)	Pond Area Determined by Gloyna Equation (ha)
Peterborough, NM			
Pond 1	3.4	61	2.9
Pond 2	2.3	40	2.1
Pond 3	2.6	37	1.9
Kilmichael, MS			
Pond 1	2.1	98	2.1
Pond 2	0.85	89	1.1
Pond 3	0.34	85	1.0
Eudora, KS			
Pond 1	3.2	95	15.3
Pond 2	1.5	82	4.2
Pond 3	3.2	87	3.3
Corinne, UT			
Pond 1	1.5	95	4.5
Pond 2	0.41	85	3.1
Pond 3	0.41	82	2.7
Pond 4	0.41	84	2.9
Pond 5	0.41	83	2.8
Pond 6	0.41	84	2.4
Pond 7	0.34	73	2.1

(ha) * 0.405 = ac

*Based on unfiltered influent and filtered effluent

according to first order reaction with the degradation constant independent of temperature and retention time; 3) No pollution losses due to seepage; 4) No settlement of influent BOD_5 as sludge. Under equilibrium conditions and neglecting seepage and evaporation losses, pond effluent quality can be predicted by the following equation:

$$P = \frac{P_i}{KR + 1} \quad . \; . \; . \; . \; . \; . \; . \quad (5)$$

in which

P = pond or effluent BOD_5 or organism concentration
P_i = influent BOD_5 or organism concentration
K = first order degradation constant (1/day)
R = detention time (days)

According to Marais, Equation 5 has been successfully used in Southern Africa to predict the reduction of fecal bacteria using a value of K equal to 2.0/day.

Model 2 is identical to Model 1, except for assumption 2), which is modified as follows: 2) Degradation rate is a first order reaction with the reaction rate dependent on temperature according to the Arrhenius equation

$$K_T = K_{T_o} \, \theta^{(T-T_o)} \quad . \; . \; . \; . \; . \; . \; . \quad (6)$$

in which

K_T = degradation constant at temperature T
K_{T_o} = degradation constant at temperature T_o
θ = temperature coefficient

The equation for Model 2 then becomes

$$P = \frac{P_i}{K_T R + 1} \quad . \; . \; . \; . \; . \; . \; . \quad (7)$$

An attempt was made to verify Model 1 and Model 2 using fecal coliform data from the primary cell of the Eudora, Kansas, and Corinne, Utah, treatment ponds. Instead of predicting effluent quality, the degradation constant, K, was determined by using the following equation

$$K = \frac{P_i - P}{PR} \quad . \; . \; . \; . \; . \; . \; . \quad (8)$$

in which

P_i = influent fecal coliform concentration (colonies/100 ml)
P = effluent fecal coliform concentration (colonies/100 ml)
R = detention time (days)

A constant value of K would verify Model 1. To verify Model 2, K_{T_o} was determined using the following equation

$$K_{T_o} = \frac{K_T}{\theta^{(T-20)}} \quad . \; . \; . \; . \; . \; . \; . \quad (9)$$

in which

θ = 1.047 (Bishop and Grenney, 1976)
K_T = value of K found in Model 1 (1/day)

A constant value of K_{T_o} would verify Model 2.

The monthly average detention time and fecal coliform influent and effluent concentrations used as input data for both models are shown for the Eudora and Corinne systems in Tables 9 and 10, respectively. Computed values of K for the Eudora system ranged from 0.31 to 2.98 with a mean of 0.91. Computed values of K_{T_o} had a mean of 1.15 and ranged from 0.44 to 2.64. Computed values of K for the Corinne system averaged 3.2 and ranged from 0.81 to 6.9. Computed values for K_{T_o} had a mean of 4.9 and ranged from 0.90 to 10.0. The computed values

Table 9. Primary cell monthly average detention time and influent and effluent fecal coliform for Eudora, Kansas.

Month	Dentention Time (Days)	Influent Fecal Coliform (colonies/100 ml)	Effluent Fecal Coliform (colonies/100 ml)
Sept.	88.9	3.7×10^6	6.2×10^4
Oct.	93.1	1.9×10^6	5.3×10^4
Nov.	94.4	7.3×10^5	2.4×10^4
Dec.	107.4	1.8×10^6	1.3×10^4
Jan.	109.2	1.3×10^6	3.0×10^4
Feb.	70.1	1.3×10^6	3.4×10^4
Mar.	74.6	9.2×10^5	1.6×10^4
Apr.	85.3	1.7×10^6	3.9×10^4
May	101.2	2.0×10^6	3.8×10^4
June	107.2	3.5×10^6	1.3×10^4
July	123.9	6.3×10^6	1.7×10^4
Aug.	116.0	3.3×10^6	2.8×10^4

Table 10. Primary cell monthly average detention time and fecal coliform influent and effluent concentrations for the Corinne, Utah, pond.

Month	Dentention Time (Days)	Influent Fecal Coliform (colonies/100 ml)	Effluent Fecal Coliform (colonies/100 ml)
Jan.	44.5	5.2×10^5	1.4×10^4
Feb.	22.7	3.0×10^5	1.1×10^4
Mar.	19.6	1.0×10^5	1.0×10^4
Apr.	23.2	3.4×10^5	4.0×10^3
May	28.5	5.7×10^5	2.9×10^3
June	27.3	3.8×10^5	2.0×10^3
July	20.7	3.4×10^5	3.9×10^3
Aug.	19.0	4.7×10^5	6.2×10^3
Sept.	21.1	8.3×10^5	1.6×10^4
Oct.	17.7	9.6×10^5	1.9×10^4
Nov.	37.2	1.1×10^6	8.4×10^3
Dec.	71.7	8.3×10^5	7.8×10^3

of K and K_{T_o} for the Eudora and Corrine systems are shown in Figures 9 and 10 respectively. The wide variation in both K and K_{T_o} reveal the limitations of using either Model 1 or Model 2 in the design of a facultative waste stabilization pond.

In contrast to the hydraulic flow assumption made by Marais, Thirumurthi (1974) states that a completely mixed flow formula should never be used for the rational design of stabilization ponds. Thirumurthi found that facultative ponds exhibit nonideal flow patterns and recommended the use of the following chemical reactor equation for pond design:

$$\frac{C_e}{C_i} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-1/2d}} \quad . \quad . \quad (10)$$

in which

C_e = effluent BOD_5 (mg/l)
C_i = influent BOD_5 (mg/l)
K = first order BOD_5 removal coefficient (1/day)
a = $\sqrt{1 + Ktd}$
t = mean detention time (days)
d = dimensionless dispersion number

Thirumurthi defined K in terms of a standard BOD_5 removal coefficient K_s using the following equation

$$K = K_s C_{Te} C_o / C_{Tox} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

in which

C_{Te} = correction factor for temperature
C_o = correction factor for organic load

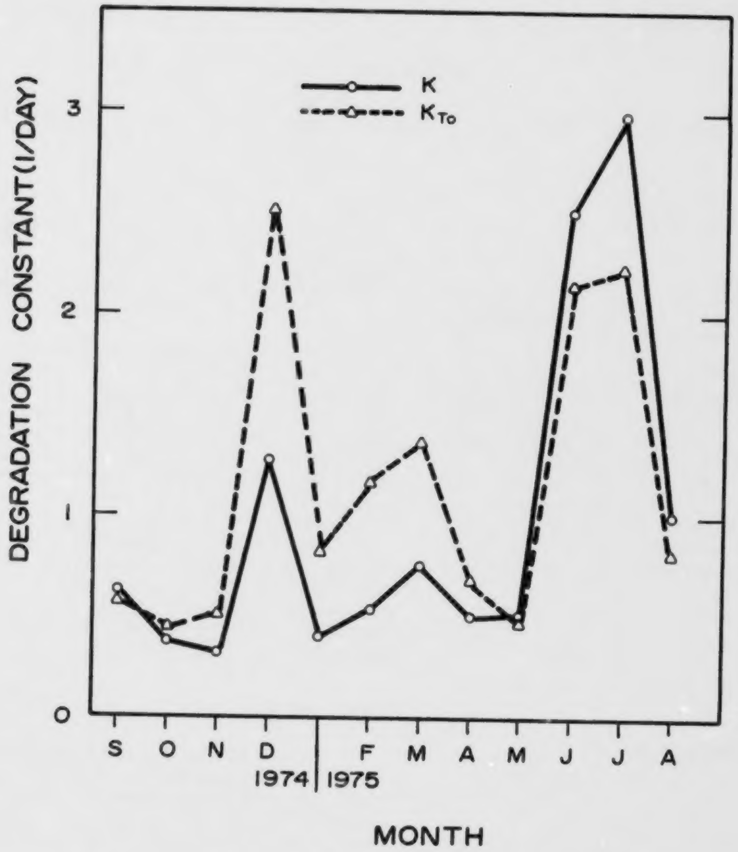

Figure 9. Computed values of K and K_{T_o} for Eudora, Kansas.

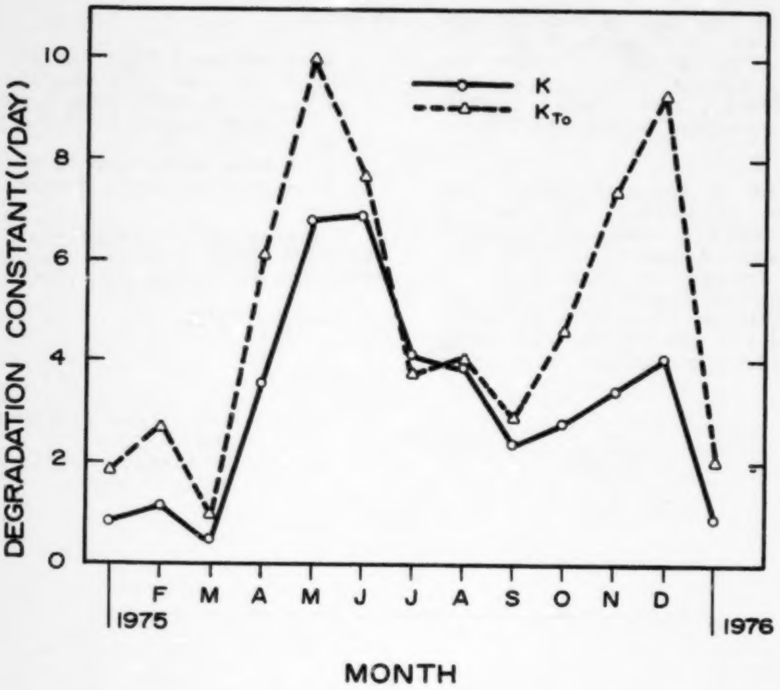

Figure 10. Calculated values of K and K_s for Corinne, Utah.

C_{Tox} = correction factor for toxic chemicals

A very narrow range of values, 0.042 to 0.071, was found for K_s. The average value of K_s was found to be 0.056/day.

Values of K_s were calculated for each cell of the Corinne, Utah, facultative stabilization pond. A narrow range of values for K_s would verify Equation 10. Using the method described by Levenspiel (1972), data from dye studies reported in EPA (1977b) were used to calculate the cell mean hydraulic detention time and dispersion coefficient. C_{Te} and C_o were calculated using the following equations.

$$C_{Te} = \theta^{T-20} \quad . \; . \; . \; . \; . \; . \; . \; (12)$$

where

θ = 1.036
T = pond temperature (°C)

and

$$C_o = 1 - \frac{0.083}{K_s} \log \frac{67.2}{L} \quad . \; . \; . \; (13)$$

where

L = organic load (kg/ha/day)

The correction for toxic chemicals was assumed to be unity.

The data used to calculate K_s for each cell are shown in Table 11. Calculated values of K_s, shown in Table 12, had a mean of 0.066/day and ranged from -0.004 to 0.127. The wide range of values found for K_s indicated the need for further refinement in the correction factors used to calculate K. Thirumurthi's assumption of nonideal flow does seem to be supported by the values found for d, the dispersion coefficient. The largest value of d was 1.71, with a d of approximately 5.0 required before Marais' completely mixed flow assumption becomes valid.

CONCLUSIONS

None of the three types of models for facultative waste stabilization pond design discussed in this paper were found to be adequate. Federal discharge standards were often violated when a pond design satisfied state organic

Table 11. Summary of data used to calculate K_s for each cell of the Corinne, Utah, facultative waste stabilization pond.

Pond Number	Month	Influent BOD_5, Cl (mg/l)	Effluent BOD_5, Cl (mg/l)	Organic Loading, L kg BOD_5/ha/day)	Mean Hydraulic Detention Time, t (days)	Dispersion Coefficient, d	Pond Temperature, T (°C)
1	Jan.	121.5	38.1	33.4	41.3	0.395	1.9
2	May	32.8	28.1	51.7	11.7	1.18	17.8
3	June-July	16.6	10.5	11.1	9.6	1.71	18.2
4	May	33.4	35.0	64.5	12.3	1.12	8.0
5	June-July	6.7	4.5	12.5	12.0	0.733	23.0
6	May	35.6	32.3	56.0	12.7	0.877	11.8
7	June-July	7.4	3.7	16.5	13.6	0.435	23.0

(kg/ha/day) * 0.893 = lb/ac/day

Table 12. Calculated values of the standard BOD removal coefficient, K_s, for the Corinne, Utah, pond.

Cell	Calculated Ks
1	0.096
2	0.025
3	0.127
4	-0.004
5	0.094
6	0.017
7	0.105

loading and hydraulic detention time design criteria. The three empirical design equations and the two kinetic design models yielded predictions not substantiated by published pond performance data. The two kinetic models differed in the basic assumptions about the hydraulic flow pattern, but neither model described the measured performance.

The hydraulic detention time is used in many of the design methods and yet very little research has been done in determining factors influencing actual hydraulic residence time. Consistent prediction of pond performance by any design method without accurate projections of hydraulic residence time is impossible. It is recommended that future research on pond performance consider the effect of physical and climatic conditions on hydraulic residence time. Once residence time can be accurately predicted, perhaps present design methods can be modified to satisfactorily predict pond performance.

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WASTE STABILIZATION POND SYSTEMS

Earnest F. Gloyna
and
Lial F. Tischler*

INTRODUCTION

Waste stabilization pond systems (WSPS) providing appropriate and economical remedies have been discussed by many engineers (Gloyna, Malina, Davis, 1976). It has been demonstrated that properly designed and operated waste stabilization pond systems are highly effective for treating wastewaters generated by the production of basic organic chemicals and related products such as synthetic polymers and resins (Engineering-Science, 1977).

TERMINOLOGY

Terminology used herein is as follows:

(1) Anaerobic Waste Stabilization Pond (AWSP) is an unmixed basin designed to remove organic materials by anaerobic biological activity.

(2) Mechanically Aerated Pond (MAP) is a basin which is mechanically mixed and aerated to allow removal of organic materials by predominantly aerobic and facultative microorganisms. Sufficient mixing and aeration is applied to maintain aerobic conditions throughout the water column and may suspend up to 200 mg/l of biomass. Anaerobic conditions may prevail in the solids which settle.

(3) Facultative Waste Stabilization Pond (FWSP) is a basin that provides an aquatic environment in which dissolved oxygen is available in the upper strata, a facultative zone exists throughout most of the depth and an anaerobic layer is present near the bottom. Aeration is provided by algal photosynthesis and natural reaeration across the air-water interface.

(4) Polishing (Aerobic) Waste Stabilization Pond (PWSP) is an unmixed, relatively shallow basin which is lightly loaded with organic material and is designed to remove organics in the wastewater. Most of the water column predominantly contains dissolved oxygen, although again, the bottom sediments may be anaerobic. This type of pond is frequently used as a polishing pond. The effluent of a multiple pond system is more uniform, and the bacterial count associated with human wastes is greatly reduced.

(5) Waste Stabilization Pond Systems (WSPS) are combinations of the above processes designed for biological treatment of wastewaters containing organic materials, Figure 1.

POND ECOLOGY

The major biological reactions which occur in waste stabilization ponds include: (a) oxidation of carbonaceous organics by aerobic bacteria, (b) nitrification of nitrogenous material by bacteria, (c) reduction of carbonaceous organics by anaerobic bacteria living in benthal deposits and bottom liquids, and (d) oxygenation of surface liquids by algae. For biodegradable wastewaters, the weight of cells produced is roughly equal to 0.5 and 0.6 times, respectively, the weight of chemical oxygen demand (COD) and biochemical oxygen demand, 5 days at 20°C (BOD_5) removed.

Methane fermentation is an essential reaction in anaerobic ponds. One of the controlling factors in an anaerobic lagoon system is the narrow pH range (6.8 to 7.2) permissible under methane fermentation. This limitation is very important since acid production must be followed immediately by methane fermentation.

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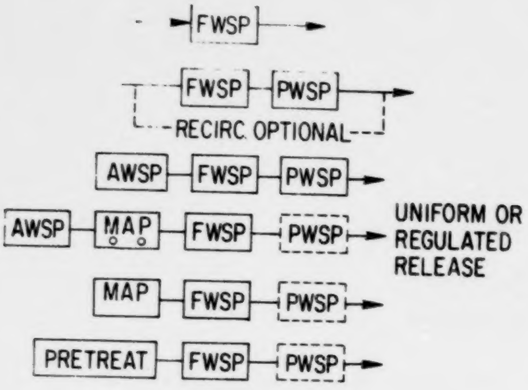

Figure 1. Possible treatment trains associated with facultative waste stabilization ponds.

A facultative pond can provide an anaerobic environment near the bottom, a buffer zone throughout the middle, and an aerobic zone near the top. The designer must remember to provide enough surface area or photosynthetic oxygen to take care of the initial soluble BOD plus that released by anaerobic decomposition and to compensate for any chemical oxygen demand.

There must be sufficient surface area and light available to accommodate the required production of photosynthetic oxygen. Similarly, the detention requirements must accommodate the rate of oxygen utilization. Significant variables and controlling parameters include illumination, nutrient, and temperature.

Radiation energy required for the production of each gram (gm) of oxygen has been determined to be about 15 x 10^3 kilograys (3,580 cal) (Oswald, 1963). One gm of cell material is theoretically equivalent to about 1.6 gms of oxygen. Assuming that algal cell substances contain 85 percent organic matter, the oxygen yield per gm of ash-included organic matter is about 1.35 gm. Oxygen production values ranging from 2.6 mg O_2/hr to 13.0 mg O_2/hr under ideal conditions have been reported (Ichimura, 1968). This level of oxygen production represents 2.6 to 12.9 gms of dissolved oxygen/m^3 per hour (7 to 35 pounds per acre foot per hour.

The overall performance of a pond system is highly temperature dependent. Sludge deposits will be degraded by anaerobic bacteria and throughout most of the pond depth the soluble BOD will be biodegraded by facultative bacteria. Thermal stratification of the pond liquid is occasionally responsible for maintaining separate aerobic and anaerobic zones for extended periods of time. Thus, designs reflect the relatively slower biodegradation rates of anaerobic-facultative systems in contrast to the more rapid rates exhibited by truly aerobic-facultative systems. The useful range of a facultative pond is 5°C to about 35°C, the lower limit being due to retardation of aerobic bacteria and algal activity. Anaerobic bacteria are not very active below 15°C. The upper limit (i.e. 35°C) is imposed by inactivation of many green algal species. If the temperatures in facultative ponds exceed 35°C, more volume must be provided or the system cooled mechanically.

INDUSTRIAL CASE HISTORIES

To illustrate the adaptability of pond systems, performance data from four waste stabilization pond systems representing an aggregate of over 10 years of operating experience and corresponding receiving water data are described. For comparative purposes, data from five activated sludge treatment plants treating mixtures of chemical and petroleum refining wastes, data from a special bench-scale study comparing activated sludge with waste stabilization pond performance, and data from exemplary plants in the Organics Chemical Development Document are utilized to compare the performance levels attainable by the two types of treatment systems (U.S. EPA, 1, 1974).

a. WSP Treatment Systems

Geographically, three of the industrial plants are located along the coastal areas of Texas, Latitude 29° and the fourth is about 400 km north. There is an abundance of sunshine and the annual precipitation varies from about 90 to 132 cm per year. The mean annual temperature (MAT) for the coastal area is about 21°C (12°C, coldest month) and for the northern site the MAT is 19°C (8.5°C, coldest month).

A variety of organic chemicals are produced and one plant also includes a petroleum refining operation. A comparison of manufacturing and waste treatment operations is presented in Table 1.

The four large waste stabilization pond systems evaluated in this study demonstrated the following:

(1) The highest long-term average effluent 5-day BOD concentration was

25 mg/l, including the algal cells in the effluent. The total influent BOD was reduced 98 to 99 percent by the three waste stabilization pond systems treating wastewaters from organic chemicals manufacturing complexes. BOD removal for the system treating mixed petroleum refining and organic chemical wastewaters was 88 percent, reflecting the lower influent BOD concentration by this system.

(2) The four waste stabilization pond systems for which total organic carbon (TOC) effluent data were available removed greater than 83 percent of the influent TOC on an average basis. The highest long-term average effluent TOC concentration for all four waste stabilization pond systems was less than 80 mg/l.

(3) Long-term average effluent COD concentrations from the three for which data were available were less than 160 mg/l.

(4) As expected, the presence of algal cells results in relatively high total suspended solids (TSS) concentrations in the waste stabilization pond effluents. The long-term average effluent TSS concentrations are as high as 100 mg/l, although they exhibit marked variability depending upon the specific characteristics of each pond system. In contrast to the TSS in high-rate biological treatment plant eflfuents, the majority of the TSS in pond effluents are algal cells.

(5) An extremely important characteristic of waste stabilization pond performance is the stability of operation. Maximum effluent variability expressed as the ratio of the maximum daily concentration to the long-term average concentration was low for BOD, COD, and TOC with the highest ratio recorded being 3.1 and with most daily variability ratios being in the range of 2.0 to 2.5. The variability of TSS was somewhat higher, reflecting algal growth in the waste stabilization pond systems. Notwithstanding this, the highest ratio of daily TSS concentrations to long-term average was 3.2.

b. WSP Data Analysis

Because the Guidelines of the U.S Environmental Protection Agency (EPA) were developed on a basis of a treatment facility designed to achieve a long-term average effluent loading, it was necessary to relate the effluent variability, both for the daily and monthly limits, to the long-term arithmetic average effluent quality. The methodology followed for the variability analyses was identical to that used by EPA, as presented in the Development Document for the Petroleum Refinery Point Source Category and its Supplement (EPA, 2, April 1974, and EPA, 3, April 1974). The basic procedure involved the following: 1) The effluent concentration data for each plant included in the investigation was plotted on log-normal probability paper; 2) the line of best fit representing approximately the upper 60 percent of the expected daily and monthly variations was determined for each plant considered; 3) a daily variability factor was defined as the ratio of that effluent concentration encompassing 99 percent of the expected daily variation to the long-term arithmetic average effluent concentration; and 4) a monthly variability was defined as the ratio of that effluent concentration encompassing 98 percent of the expected monthly variation to the long-term arithmetic average effluent concentration.

Table 2 summarizes the case history data for the four waste stabilization pond treatment facilities. The effluent quality from the waste stabilization ponds in terms of BOD is comparable to other biological treatment processes. However, COD and TOC reductions obtained by pond systems are often much better. This is attributable to the extended hydraulic detention in pond systems which provide the microorganisms sufficient time to degrade the more refractory compounds. Residual organics may be readily measured by COD or TOC tests, but short-term BOD tests may not provide accurate information. The variability of TSS in these effluents represent the algal growth cycles which occur as a function of incident solar energy and climatological phenomena.

The variability factors summarized in Table 2 are attempts to account for the "inherent variability" in the long-term average effluent concentration from a properly designed and operated waste treatment plant. The term "inherent variability" is defined as that variance in effluent quality attributable to the basic nature of the treatment processes, the characteristics of the wastewater, and the climatological conditions--none of which can be significantly altered by externally applied changes. The published EPA effluent limitations are based on maximum allowable daily and monthly average discharges. Because treatment system design and performance analysis has been on more of an "average" or steady-state basis, it becomes mandatory to consider the variability in the treatment system's effluent quality in order to determine its compliance with the federal limitations.

Table 1. Industrial waste stabilization pond systems.

Subject Itemization	Characterization of Industry and Treatment Plant			
	A	B	C	D
Major Production	Ethylene Proplene Acetylene Ethyl Benzene Ethylene Oxide Ethylene Glycol Axo-Alcohols Amino Polyols Styrene Butadiene Cellulose Acetate	Olefins Ethanol Acetyldehyde Ethylene Glycol Oxo Chemicals Polyproplene Polyethylene	Hexane Cyclohexane Benzene Toluene P-Xylene O-Xylene Paraffins Propylene Isobutylene Naptha Ethane Ethylene Solvents Motor Fuel Lubricants	Polyethylene Ethylene Vinyl Acetate Copolymer High Density Polyethylene Adiponitrile Hexamethylene-diamine Ethylene Methanol Adipic Acid Hydrogen Cyanide Nitric Acid Ethylene Copolymers
Pond System				
1. Mech. Aerated	No. = 1 Ar. = 3.2 ha (7.8 ac) Deten. = 1 d Depth = 2.5 m	-0-	-0-	-0-
2. Anaerobic a) 1st Stage	No. = 5 para. Ar. = 16.2 ha (40 ac) Deten. = 50 d	No. = 7 ser. Ar. = 25 ha Deten. = 60 d Depth = 2 m	-0- -0-	No. = 1 Ar. = 16.2 ha (40 ac) Deten. = 30 d Depth = var.
b) 2nd Stage (addl. proc. water, neut., etc.)	No. = 1 Ar. = 56.7 ha (140 ac) Deten. = 60 d Depth = 1.5 m	-0-	-0-	No. = 1 Ar. = 2.2 ha (5.5 ac)
3. Facultative	No. = 1 Ar. = 118 ha (291 ac) Deten. = 68 d Depth = 1 m	No. = 2 ser. Ar. = 174 ha (430 ac) (180, 250) Deten. = 180 d Depth = 1-2 m	No. = 3 ser. Ar. = 142 ha (350 ac) (175, 70, 105) Deten. = 14 d Depth = 1.3 m	No. = 1 Ar. = 51.4 ha (127 ac) Deten. ≈ 17 d Depth = 1 m
Discharge	Continuous Barge Canal	Seasonal-winter Oxygen sag Control in river	Continuous brackish water	Continuous River
Treatment Parameter				
Effluent				
BOD_5 (mg/l)	24(1.8)[a]		8(1.6)[a]	16(2.1)[a]
COD (mg/l)	158(1.4)		97(1.3)	--
TOC (mg/l)	101(1.9)		37(1.3)	29(1.6)
TSS (mg/l)	80(1.5)		26(1.8)	80(2.7)
Average Monthly Removal				
BOD_5 (%)	98	99(2.4)	92	98(2.1)
COD (%)	93	--	68	--
TOC (%)	89	93(2.1)	59	93(2.7)

[a] Variability factor.

Table 2. Summary of waste stabilization pond treatment performance in the organic chemicals and petroleum refining industries.

Plant	BOD			COD			TSS			TOC		
	Var. Ratio Day Max.	Var. Ratio Mo. Av.	Long-term Av. Conc. (mg/l)	Var. Ratio Day Max.	Var. Ratio Mo. Av.	Long-term Av. Conc. (mg/l)	Var. Ratio Day Max.	Var. Ratio Mo. Av.	Long-term Av. Conc. (mg/l)	Var. Ratio Day Max.	Var. Ratio Mo. Av.	Long-term Av. Conc. (mg/l)
A	2.4	1.8	24	2.5	1.4	158	3.2	1.9	101	2.0	1.5	80
B	3.1	2.4	15	2.0	2.0	134	2.6	1.9	46	-	2.1	56
C	1.8	1.6	8	1.4	1.3	97	2.2	1.8	26	-	1.3	37
D	-	2.1	16	-	-	-	-	2.7	80	-	1.6	28
EPA BPT Guidelines:												
Organ. Chem. Indus.[a]	4.5	2.0	20-30	-	-	Varies	4.5	2.0	30	-	-	-
Petrol. Ref. Indus.[b]	3.2	1.7	15	3.1	1.6	80-110	3.3[c]	2.1[c]	10	-	-	-

[a] As amended 5/12/76 (40 CFR 414, 41 FR 19310), Remanded 4/1/76 (40 CFR 414, 41 FR 13936).

[b] Data from 308 Questionnaire raw data summary sheets, January 1975 through September 1976.

[c] As amended 40 FR 21939, 5/20/75.

Note: BOD = Biochemical Oxygen Demand, 5-day, 20°C;
TSS = Total Suspended Solids;
Var = Variability; Day = Daily; Mo. = Monthly; Av. = Average Max. = Maximum; Conc. = Concentration.

There are several points which should be kept in mind when considering the variability of the effluent quality from any particular treatment system. In general, as the effluent concentration or loading from a treatment system is decreased, the effluent variability expressed as a ratio increases. This phenomenon is primarily due to the fact that the distribution of effluent concentrations or loadings is bounded at the lower end by the non-removable portion of a particular constituent and by the sensitivity of the analytical tests involved. There is no similar boundary on the upper end of effluent concentrations. For example, a 15 mg/l variation in BOD when the long-term effluent concentration is 15 mg/l variation represents a variability factor of 2.0, whereas the same 15 mg/l variation represents a variability factor of 4.0 if the long-term average is 5.0 mg/l.

Another observation concerning the variability factors developed as a result of this study is the fact that the choice of a variability factor encompassing 99 percent of the expected variation implies that the maximum daily effluent loading predicted on this basis may be expected to be exceeded 1 percent of the time. Likewise, the maximum monthly average may be exceeded 1 month out of every 50 months of operation if the 98 percentile is used. Such occurrences are inherent in a probabilistic approach to effluent variability and present no major problems with respect to design, provided they are properly considered. Variability analyses are shown in Figures 2 through 5.

c. WSP Model

Typical treatment models have been developed for these pond systems and the presentation is in general accordance with the format used in the U.S. EPA Development Document. These models should not be used for general design purposes for these observations apply only to a unique grouping of industrial wastewaters, climate, and operations. Only one example (FWSP) is presented herein:

Facultative Waste Stabilization Pond (FWSP) - ("Aerobic" under EPA definition, but in reality this is a misnomer.)

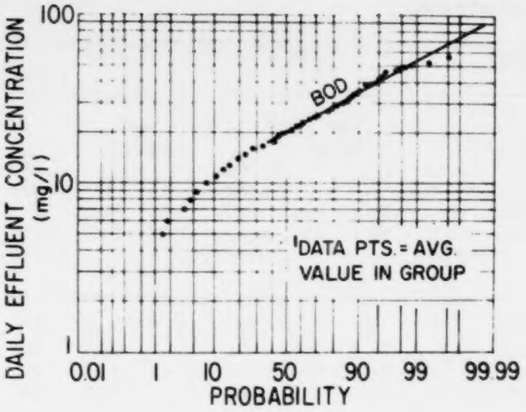

Figure 2. Daily maximum variability analysis.

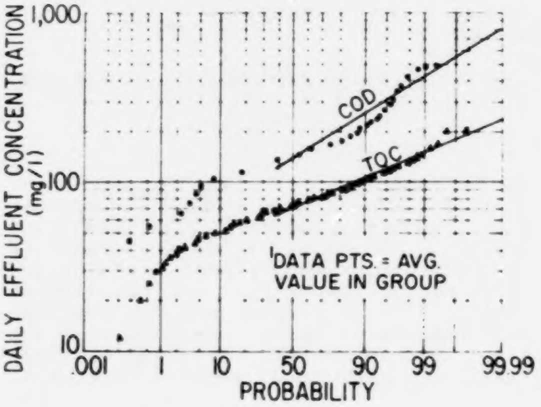

Figure 3. Daily maximum variability analysis.

EFFECTIVENESS:

BOD Removal: Percentage removal depends upon design. Soluble BOD_5 in effluent is less than 10 mg/l in most applications. Total effluent BOD_5, including TSS contribution due to algal cells, is less than 25 mg/l long-term average.

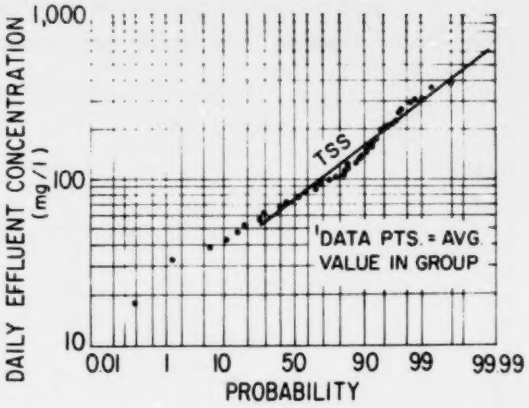

Figure 4. Daily maximum variability analysis.

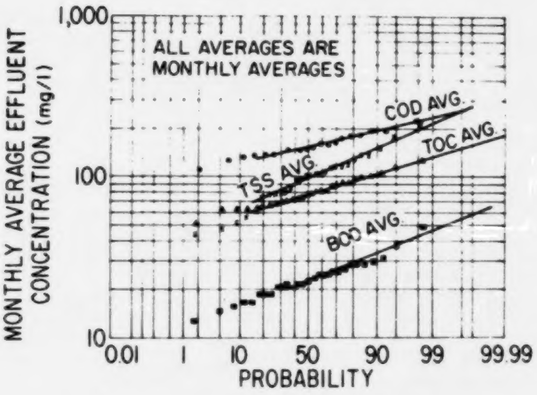

Figure 5. Monthly averge variability analysis.

Effluent BOD_5: Long-term average soluble fraction approaches 10 mg/l. Suspended solids fraction is approximately 0.2 kg of BOD_5 per kg TSS in the effluent.

Phenol Removal: Greater than 99 percent of phenolic material is removed at specified design criteria.

Phenol in effluent is less than 0.1 mg/l long-term average.

Oil Removal: Oil removal is greater than 90 percent. Attainable effluent concentration is less than 5 mg/l. It should be noted that chlorophyll in algal cells will be measured in most oil and grease analyses.

COD, TOC, and TOD Removal: Up to 90 percent TOC removal, has been obtained, depending on design of system. When preceded by anaerobic or mechanically aerated lagoons, typical performance is 45 to 80 percent TOC removal. COD and TOD removal data are not as complete as for TOC, but indicate that a conservative assumption is that COD and TOC removals are equal to TOC removal. When used as a polishing step behind activated sludge, ponds are usually not designed for TOC removal greater than 50 percent. It should be emphasized that the above removals are on a total TOC, COD, and TOD basis, and include the contribution of algal cells to effluent organic concentrations.

APPLICATION LIMITS:

pH	5-10	
Temperature	$\geq 55^{\circ}F$	(Annual mean daily minimum temperature as presented in the Decennial Census of U.S. Climate, U.S. Weather Bureau)
Oil	≤ 35 mg/l	
TDS	$\leq 10{,}000$ mg/l	
TSS	≤ 125 mg/l (annual average)	
Sulfates	$\leq 1{,}500$ mg/l	
Sulfides	≤ 200 mg/l	

OPERATING BASIS:

BOD_5 Removal:

$$S_e = 0.73\ (S_o/t) + 0.43\ (L) + 1.0$$

in which

S_e = effluent BOD_5 concentration (soluble and solids), mg/l
S_o = influent BOD_5 concentration (soluble and solids), mg/l
t = hydraulic detention time, days, and
L = surface loading, kg BOD_5/ha/day (lb BOD_5/acre/day)
$S_o \leq 3{,}500$ mg/l
$t \geq 10$ days
$L \leq 67$ kg BOD_5/ha/day (60 lb BOD_5/acre/day)

TOC Removal:

$$S_e = S_o\ [(1) - [13.5\ (\log_e t) - 0.1\ (L) + 10]/[100]\ ]$$

in which

S_e = effluent total TOC concentration (soluble and solids), mg/l,
S_o = influent total TOC concentration (soluble and solids), mg/l,
t = hydraulic detention time, days, and
L = surface loading, kg TOC/ha/day
$S_o \leq 2{,}000$ mg/l
$t \geq 10$ days
$L \leq 50$ kg TOC/ha/day (45 lb TOC/acre/day)

Effluent TSS (predominantly algal cells):

$$X_e = 32\ (\log_e L) + 27\ (\log_e t) - 100$$

in which

X_e = annual average effluent TSS, mg/l,
L = surface loading, kg BOD_5/ha/day, and
t = hydraulic detention time, days
$L \leq 112$ kg BOD_5/ha/day (100 lb BOD_5/acre/day)
$t \geq 10$ days

Effluent Soluble Organics:

Soluble BOD_5 = S_e(total BOD_5) $- 0.2\ X_e$
Soluble TOC = S_e(total TOC) $- 0.2\ X_e$
Soluble COD = S_e(total COD) $- 0.9\ X_e$

TEMPERATURE:

Within the specified temperature application criterion, these specific design equations do not need to be corrected for winter conditions. The water temperature in these ponds will be essentially the same as monthly average temperature. (Note: This specific temperature criterion is not general because colder, seasonal temperatures may control pond designs.)

COST PARAMETER:

Flowrate.

COST CURVE SCALE FACTOR:

Surface loading of BOD_5 or TOC.

RESIDUES:

Solids accumulation is low if specified influent TSS requirement is met. It can be assumed that 30 cm of depth will provide over 10 years of solids storage.

MAJOR EQUIPMENT:

FWSP Ponds--In these examples parallel construction with equal distribution of volume is generally used.

Construction:	Operating depth 1 to 2 m plus 0.5 m of solids storage; dikes with side slopes between 1 to 3 (vertical to horizontal) and 1 to 6, top width 6 to 8 feet; freeboard 1 m above maximum water level; erosion protection; permeability of bottom and sides less that 10^{-5} cm/sec.
Inlets/Outlets:	Gravity flow, submerged outlet or provides with baffle and weir. Splitter box is required.
Nutrient Storage and Feed:	As required.

d. WSP Comparison with Activated Sludge

The waste stabilization pond performance data described above were compared with long-term operating records obtained from five activated sludge plants treating similar wastewaters and one set of detailed laboratory studies. These data were obtained and evaluated in order to provide a better data base than was available from the Organic Chemicals Development Document which was used by EPA to establish the remanded organic chemicals effluent guidelines. Comparison of the waste stabilization pond performance data summarized above with the long-term operating data from these activated sludge plants, data from the Organic Chemicals Development Document, and bench-scale treatability data for the wastewaters from one of the participating plants in this evaluation demonstrates the following:

(1) Total BOD removals, including the suspended solids contribution, by the waste stabilization pond systems are at least as good as, if not better than, the high-rate biological treatment systems.

(2) The long-term average effluent concentrations of refractory organic materials (COD and TOC) in the waste stabilization pond system effluents are 50 percent less than the corresponding values for the high-rate biological treatment systems.

(3) The variability in effluent quality from the waste stabilization pond systems is lower for all waste constituents, including total suspended solids, than the high-rate biological treatment systems.

(4) The average effluent TSS concentration from the waste stabilization pond systems is higher than the corresponding value from the five activated sludge case histories, but lower than the effluent TSS values exhibited by the exemplary plants in EPA's Organic Chemical Development Document.

(5) Overall, the performance of the waste stabilization pond systems treating organic chemicals and petroleum refinery wastewaters equals or, as in the case of TOC and COD, exceeds performance exhibited by high-rate biological treatment systems. The only possible exception to this is effluent TSS concentrations which consist principally of algal cells in the waste stabilization pond effluents. It is thus important to consider the impact of these algal cells on receiving waters to demonstrate any necessity for removing algae before final discharge. Table 3 summarizes various comparisons.

Bench-scale studies conducted with wastes from Plant B provided the data

Table 3. Comparison of activated sludge and waste stabilization pond effluents.

	BOD				COD			
	Rem. %	Av. Effl. Conc. (mg/l)	Day Max. Var.	Mo. Av. Var.	Rem. %	Av. Effl. Conc. (mg/l)	Day Max. Var.	Mo. Av. Var.
Waste Stabilization Pond:								
Plant A	98	24	2.4	1.8	93	158	2.5	1.4
Plant B	99	15	3.1	2.4	-	134	2.0	2.0
Plant C	92	8	1.8	1.6	68	97	1.4	1.3
Plant D	98	16	-	2.1	-	-	-	-
Activated Sludge:								
Case 1	-	34	6.6	2.2	-	334	2.5	1.3
Case 2	-	16	18.9	3.0	-	229	8.7	1.7
Case 3[a]	-	24	3.0	1.7	-	333	2.0	1.3
Case 4[b]	-	15	4.0	2.0	-	110	1.8	1.6
Case 5	-	16	2.8	2.1	-	237	2.4	1.8
Bench Scale, Plant B	97	30	-	-	79	396	-	-
EPA Exemplary Plants (All)	93	82	-	-	74	378	-	-
EPA Exemplary Single-Stage Plants	92	-	-	-	69	-	-	-
EPA Guidelines:								
Organ. Chem. Indus.-BPT[c]	-	20-30	4.5	2.0	-	Varies	-	-
Organ. Chem. Indus.-BAT[c]	-	20-30	3.9	2.1	-	Varies	3.9	2.1
Petrol. Ref. Indus.-BPT	-	15	3.2	1.7	-	80-110	3.1	1.6
Petrol. Ref. Indus.-BAT[c]	-	5	2.1	1.7	-	20-27	2.0	1.6

[a] Includes refining wastewaters.
[b] Hybrid plant, i.e., biological and clean stream effluents combined in holding/polishing pond before final discharge.
[c] Remanded by Federal Courts by EPA.

Note: BOD = Biochemical Oxygen Demand, 5-day, 20°C;
COD = Chemical Oxygen Demand;
Av. = Average; Day - Daily; Mo. = Month; Effl. = Effluent; Max. = Maximum;
Rem = Removal; Conc. = Concentration; Var. = Variability.

shown in Table 4. Note that after 24 hours, there was little improvement in the soluble organic quality attainable by the activated sludge system. Also, the data show the impact obtainable through the use of anaerobic and facultative ponds. Removals of BOD as high as 97 percent were obtained with an influent BOD of nearly 1,000 mg/l. COD and TOC removals were also high at 79 and 80 percent, respectively.

It is particularly important to note that the TSS concentrations used by the EPA to define 1977 Effluent Limitations for the Organic Chemical Industry were exceeded by the vast majority of the exemplary plants used by the EPA in their analysis. The TSS concentrations in the pond effluents, consisting mainly of algae, are consistent with those that were obtained in the EPA survey, but are not consistent with the limitations that were finally established in the Effluent Guidelines.

EMPIRICAL DESIGNS

During the last decade, various authors have reported on the performance of various pond systems (Stander and Meiring, 1962; Aguirre and Gloyna, 1970). While some observations have been based on long-term studies, many conclusions appear to have been based on relatively short term tests. Because of the long-term detention, environmental effects and cyclic characteristics of algae, it is important to base conclusions on fairly lengthly studies. Important design parameters must include information on variability of waste characteristics, environmental factors, and effluent requirements.

The literature contains many references to FWSP designs, including generalization based on population served per volume, fixed detention and organic load per surface area. Interestingly, while the pond system is exceedingly complex, experience in pond design based on a

Table 4. Activated sludge treatment of wastewater from Plant B.

	Detention Time			
	Initial	One-Day	Two-Day	Three-Day
MLSS, mg/l	4,070	4,520	4,620	4,470
MLVSS, mg/l	3,040	3,540	3,580	3,390
COD, mg/l	1,674	432	444	470
TOC, mg/l	600	110	120	155

given geographical area and wastewater may produce best treatment results.

Clearly, major factors important in the design of a FWSP must include the total BOD (soluble and settleable load), flow, temperature (based on average temperature of most critical month, hot or cold), light (although a precise knowledge of the total flux is academic), total potential sulfide concentration, and an understanding of potential for chlorophyll inhibition (Gloyna, 1971).

One empirical relationship for design of facultative ponds has been developed over years of laboratory, pilot plant, and field experience. An example follows:

Given:

A biodegradable wastewater:

BOD_5 influent, 20°C = 250 mg/l
BOD_u influent = 305 mg/l
Reaction coefficient (Base e)
= $K_{20°C} = 0.35\ day^{-1}$
Flow = 7.57×10^6 cu l/day (2×10^6 gal/d) (U.S.)
Design temperature
= 25°C (average coldest month)

Assume:

1. Evaporation rate = rainfall
2. Percolation rate = negligible
3. Sunlight = usually sunshine
4. No chlorophyll inhibition problems, f = 1 (Huang and Gloyna, 1968)
5. Sulfates < 500 mg/l, f' = 1 (Gloyna and Espino, 1969)
6. Solids typical of domestic wastes

Required:

Calculations showing (a) organic load, (b) volume, (c) depth, (d) surface area, (e) detention, and (f) θ surface loading for BOD_u if = 1.085.

Solution:

(a) Organic Load = (305 mg/l) (7.57×10^6 cu l/day)
= 2309 kg BOD/d (5091 lb BOD_u/day)
(b) Volume = $3.5 \times 10^{-5}\ QL_a\ [1.085^{(35-T)}]$ (1) (1)
= (3.5×10^{-5}) (7.57×10^6) (305) $[1.085^{(10)}]$
= (3.5×10^{-5}) (7.57×10^6) (305) (2.26)
= 183,000 cu m (148 acre ft) exclusive of sludge storage)
(c) Depth (Use) = 1.25 m (4.1 ft)
(d) Surface Area = 18.3 ha (45.2 acres) (based on 1 meter effective depth and 0.25 m sludge storage)
(e) Detention = 24 days (based on 1 meter effective depth)
(f) Surface Loading = (2309 kg/day)/(18.3 ha)
= 126 kg BOD_u/ha per day (112 lb BOD_u/acre per day)

The BOD_5 removal efficiency can be expected to be 80 to 90 percent or better. The efficiency based on unfiltered effluent samples can be expected to vary unless a maturation pond is used as a follow-up unit.

The detention time provided by the above equations is given in Figure 6. Added detention times may be provided by increasing the depth, but this extra depth must not be used to calculate the surface area.

The minimum depth of about 1 meter is required to control potential growth of emergent vegetation. If the depth is too great, there will be inadequate surface area to support photosynthetic action. Deep ponds tend to stratify during hot periods. The following design guidelines for depth are suggested:

CASE	DEPTH	RELATED CONDITIONS
1	1 meter	Generally ideal condition, very uniform temperature, tropical to subtropical, minimum settleable solids.
2	1.25 meters	Same as above but with modest amounts of settleable solids. Surface design based on 1 m depth and 0.25 m used for sludge. (For wastes containing considerable amounts of biodegradable, settleable solids, the FWSP should be preceded by an anaerobic pond.)

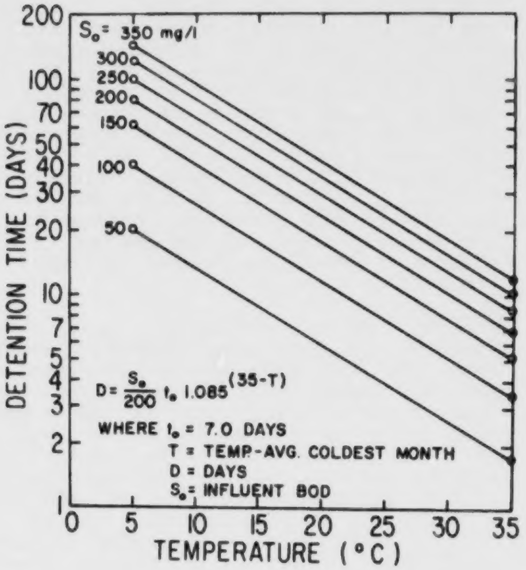

Figure 6. Detention for facultative waste stabilization pond.

3	1.5 meters	Same as Case 2 except for significant seasonal variation in temperature, major fluctuations in daily flow. Surface design based on 1 m of depth.
4	1.5 to 2 meters and greater	For soluble wastewaters that are slowly biodegradable and retention is controlling. Surface design based on 1 of depth.

WSP IMPACT ON WATERWAYS

There are three processes that have to be considered in examining the effect of pond algae on a receiving water. First, over a relatively long time period, some of the pond algae will die and exert an oxygen demand on the oxygen resources in the receiving stream over a large segment of its length. Second, introduced algae will adjust their generation rates within a matter of days to reflect the new set of environmental constraints and exert a negligible demand on the oxygen resources on the receiving body of water. Third, the resident algal populations may reflect a downstream increase in size as a function of nutrient loadings, if the receiving system is not light-limiting at the autotrophic level.

Ample evidence indicates that placement of algae into a dark environment does not cause their immediate death. Rather, algae die at a diminishing rate and require 20 to 30 days to deplete their nutritional reserves. Studies indicate that at least some cells of Chlorella and Scenedesmus remained sufficiently viable to be recultured after 60 to 90 days in an unheated anaerobic digester (Golueke, Oswald, and Gotaas, 1957).

The most common cause of algal death in effluents derived from FWSP occurs during the chlorination process. It has been shown that it is possible to chlorinate at a dosage to kill E. coli and permit survival of certain algae. The maximum dosage permissible for algal survival appears to be about 8 to 10 mg/l. Above this dosage, lysis of algal cell increases and the BOD of the effluent increases accordingly (Hom, 1968).

The respiration requirements of viable algae are only about 3 percent of the body weight. If algae discharged to a stream remain suspended in light and are supplied the needed nutrients, they will grow and produce excess oxygen in the absence of severe salinity and temperature stresses. Data collected by Plant A illustrate that the algae can withstand major changes in salinity and temperature. If the algae settle out, they will respire at a low rate, with oxygen demands of dead algae being approximately 1.4 times the dry weight of freshly destroyed algal cells.

Algae are important natural residents of stabilization pond effluents, as well as the receiving waters into which they are discharged. Analyses substantiate that in a properly designed and operated pond system, algal cells in the effluent do not constitute a significant impact on the dissolved oxygen resources of the receiving waters. Furthermore, algae usually serve as an important link in the receiving water food chain. Discharge of algal cells in a properly treated effluent may increase productivity at higher trophic levels of aquatic organisms of economic importance, for example, fin fish, shrimp and crab in estuarine environments. Therefore, algae do not appear to effect most receiving waters in a deleterious manner and their removal from a highly treated effluent should not be required unless studies of the receiving waters prove this step to be necessary.

The removal of algal cells from pond effluents has been studied in considerable detail (Middlebrooks, 1974; Parker, 1976; Dinges, 1978; and Bare, Jones, and

Middlebrooks, 1975). The removal systems include the use of predators and water hyacinth cultures, air flotation, sand filtration, rock filtration, and nutrient control.

ECONOMIC CONSIDERATIONS

While the operational efficiency of certain pond systems in terms of attainable effluent quality characteristics can be clearly demonstrated, there are factors which support the implementation of these systems as Best Practical Control Technology Currently Available and/or Best Conventional Pollutant Control Technology for treatment of municipal wastewaters and certain industrial waste streams. Table 5 shows estimated energy requirements for two "high-rate" biological treatment systems, an aerated lagoon, and a waste stabilization system, all designed to meet the same effluent BOD for a given influent condition. The analysis excludes any waste pretreatment requirements and tertiary treatment. Pretreatment needs are essentially the same for each case, although the activated sludge system would require equalization. The waste stabilization system is more economical than the other biological treatment systems in all respects, and the energy savings are substantial. If electricity is provided by a fossil-fuel-fired power plant, the energy saved by a waste stabilization pond, as compared to an activated sludge plant, for the example in Table 5 represents about 136 tonnes (150 tons) per year of coal. If equal COD or TOC removal were used as the design criterion, then the energy requirements would be much higher for the other systems treating a refractory chemical wastewater since considerable additional hydraulic detention time (or disk area in the case of the rotating biological contactor) would have to be provided. Similarly, if lime is used to assist certain chemical treatment requirements, about 6.3 kilojoules (≈ 6 million Btu) will be needed to produce a 1,000 kg of lime.

Table 5. Comparison of energy requirements for biological treatment systems.[a]

Treatment Type	Energy Use (kwh/yr)[b]
Activated Sludge	1,000,000
Aerated Lagoon	850,000
Rotating Biological Contactor	120,000
Waste Stabilization Pond	-0-

[a] Basis:

1. Flow - 3,785 cu. meters per day (1 million gallons U.S. per day).
2. Influent BOD_5 - 350 mg/l (2,900 pounds per day).
3. BOD removal rate - 0.001 mg/l per day.
4. Excludes pumping and pretreatment costs.

[b] To convert from kilowatt hour to joule, multiply by 3.6×10^6.

Biological treatment processes operate at peak performance levels when the volume and composition of influent wastewaters are kept constant. This is particularly true in the treatment of organic chemical wastewaters which are highly variable in quantity and quality and which can be very detrimental to continuous operation of a high-rate biological treatment system. Because of this, all BPCTCA type biological systems which rely on high-rate biological performance require considerable equalization capacity to assure that shock loadings are minimized. Wastewater stabilization ponds provide considerably more equalization and their large volumes can accept short-term fluctuations and shock loads with much greater efficiency than can the high-rate biological systems (Gloyna, Herring, and Ford, 1970). Waste stabilization pond systems which are used in the organic chemicals industry generally provide a residence time of 100 days or more. Retention times for equalization of this order of magnitude cannot be economically provided in the typical high-rate biological waste treatment system.

SUMMARY

Wastewater stabilization ponds, when properly designed and operated, and when located in a region with a climate suitable to their maximum efficiency of operation, provide effluent quality evels of treated organic wastewaters completely consistent with Best Practicable Control Technology Currently Available as defined by the EPA in their Development Documents for both the Organic Chemicals and Petroleum Refining industries. In fact, the soluble BOD removals of refractory COD are often considerably better for a well-operated and well-designed wastewater stabilization pond system than those obtainable using high-rate biological treatment facilities. The major problem which accrues with wastewater stabilization ponds is in meeting the total suspended solids limitations established by the applicable Effluent Guidelines.

The engineering approach to the solution of any problem must take into account the technical alternatives. In a WSPS design, it is necessary that each facility be a custom design having adjustments for many factors associated with a local situation. Properly designed and applied integrated WSPS will permit maximization of the reclamation potential for both wastewater and nutrients.

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DESIGN AND CONSTRUCTION OF WASTEWATER STABILIZATION PONDS

Earl C. Reynolds, Jr. and
Scott B. Ahlstrom*

In late 1967, construction was completed on a centralized sewage treatment facility for the City of Logan, Utah, consisting of a series of stabilization ponds and a chlorination structure. Design and services during construction on this project were accomplished by CH2M HILL. This presentation will discuss CH2M HILL's experience in the design and construction of stabilization pond systems. Our purpose will be to identify particularly critical problem areas; and some of the techniques that CH2M HILL has utilized to solve these problems.

This discussion on stabilization pond design and construction will focus on the Logan system. This is appropriate since the ponds serve our hosts, the City of Logan and Utah State University. Participants at this conference may have an opportunity to view these stabilization ponds during their stay in Logan. Experience from other stabilization pond systems also will be utilized to illustrate design and construction practices.

The first step in the design of a waste treatment facility involves establishing the proper design criteria. Many of the design parameters pertaining to wastewater stabilization ponds are established by state or other regulatory agencies. This presentation will briefly identify some of these standards, but will be directed principally to stabilization pond design and construction as experienced by CH2M HILL.

REGULATORY STANDARDS

Design standards are formulated by regulatory agencies to ensure that the design of sewage treatment systems is consistent with U.S. Public Health and water quality objectives. The Ten-State Standards have formed the basis for regulatory agency design criteria for stabilization ponds for a number of years and have done so quite satisfactorily. They provide guidance in numerous areas including design loadings, physical location, hydraulic capacity, and construction details. The Utah state standards are similar in some respect, although they do depart in significant instances from the Ten-State Standards.

Some of the principal points covered under the Utah state standards are as follows:

> Isolation, or the necessity for the treatment facility to be set apart from habitation and water supply sources.
>
> Number and size of the stabilization pond compartments with due consideration given to loading, effluent disinfection, and future population growth.
>
> Embankment and dike design, construction, and protection.
>
> Pond bottom treatment to provide for suitable drainage, uniform depth, and leakage prevention.
>
> Inlet and outlet structures which minimize short circuiting and prevent leakage.
>
> Flow measurement.

The U.S. EPA and the Federal Water Pollution Control Acts (PL 92-500 and PL 95-217) also affect the design and construction of wastewater stabilization ponds. Specific procedures must be complied with in order to obtain a federal construction grant. Furthermore, federally approved discharge standards must be satisfied.

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DESIGN AND CONSTRUCTION

This discussion of stabilization pond design and construction has been organized into the following principal elements.

Site Reconnaissance
Geotechnical Investigation
Aeration
Layout
Earthwork
Dike Protection and Seepage Control
Hydraulics
Structures, and
Leakage Test

Site Reconnaissance

The fact that a stabilization pond is built principally from materials available at the selected site focuses particular attention on site selection. Land availability and land suitability must be assessed. Topographical features should be observed and undesirable geological characteristics such as basalt outcrops, granular alluvial deposits, and limestone formations noted. The site should be relatively flat and the soil preferably should be a fine textured, clayey type rather than coarse granular material. One extremely important factor which must be considered is the availability of the land, and the ability to obtain it without arousing the concern of surrounding property owners. Ideally, the site should be located below the city, close to a receiving stream, safe from flooding, and available for purchase at a reasonable cost.

Based on the information gathered during the site reconnaissance regarding land availability, topography, geology, and other miscellaneous inputs, a preliminary site layout can be developed.

The site selected for the Logan stabilization ponds is located in the bed of the ancient Great Salt Lake--an excellent location for a facility of this type. Approximately half of the site was farmed. The other half consisted of hummocky marshland utilized for late summer pasture.

Geotechnical Investigation

After developing a preliminary site layout, confirm that suitable soil conditions exist. Test pits and borings should be strategically located and soil samples obtained. Laboratory tests conducted on the soil samples must determine physical classification, strength, and permeability. From this information, a subsurface profile is developed and geological conditions are confirmed. Sources of materials for lining, riprap, and embankments should also be located.

The geotechnical investigation at Logan identified the material at the site as a highly plastic clay. Although a 500-acre site was involved, very few test holes were required due to the general uniformity of the geological conditions over the bed of the ancient Great Salt Lake. In other less uniform sites a test hole every acre may be far from adequate. For example, in Idaho a stabilization pond was recently constructed on a site which was underlain with fractured and highly pervious basaltic lava flows. A great deal of difficulty was encountered during construction because the undulating nature of the surface of the lava flows had not been adequately identified. Rock was encountered during excavation, causing substantial cost over-run and raising difficult questions concerning prevention of leakage.

In parts of the country, sites suitable from a topographical standpoint are underlain with limestone which is subject to caverns and fissures, which serve as underground conduits. In these cases a thorough geological study would be a necessity.

In other instances, and this is not unusual in the arid west, we have found that layers of caliche underlie many of the suitable sites. In one instance, we determined that this caliche would cause great difficulties if a winter storage lagoon for an industrial waste treatment operation were constructed. The industrial waste would become anaerobic during winter storage, and seepage of the anaerobic fluid from the pond would be expected to dissolve the caliche, endangering the embankment structure and/or causing large-scale leakage.

Once the geotechnical engineering has been completed, a final location for the stabilization pond can be recommended. An integral part of this recommendation involves choosing a site which complies with the governing state regulations. If the site recommended satisfies the desires of the community, a final site layout is developed and the land acquisition process set in motion.

The site selected for Logan was west of town and involved about 1 1/2 miles of interceptor sewer to conduct the waste from the city to the stabilization ponds. By elevating the interceptor sewer on a compacted berm, all of the waste from the City of Logan could reach and pass through the ponds by gravity. Ideal soil conditions on the site indicated that potential pond leakage would be minimal. Not only was the soil adequate to seal the ponds, but the aquifer beneath the ponds existed under a positive hydrostatic head. Two flowing artesian wells had to be plugged at the site prior to construction.

Aeration

Aerobic and facultative ponds may derive the oxygen necessary for proper operation from natural or mechanical means. Under natural means, dissolved oxygen is furnished by oxygen transfer between the air and water surface and by photosynthetic algae. The amount of oxygen supplied by natural surface reaeration depends largely on wind-induced turbulence--a supply mechanism usually neglected in design because of its lack of dependability. The primary source of natural aeration is from algal photosynthesis. The photosynthetic production of oxygen is cyclic. During a sunny day the liquid contents of a shallow lagoon will become supersaturated with oxygen. Photosynthesis ceases at night, but respiration continues, resulting in an increase in carbon dioxide and a decrease in oxygen concentration.(1)

In a mechanically aerated lagoon the major portion of oxygen transfer is caused by the aeration device. Two basic methods of aerating sewage are (1) to agitate the sewage mechanically at the surface so as to promote solution of air from the atmosphere, or (2) to introduce air or pure oxygen into the sewage with submerged porous diffusers or air nozzles.

In the case of surface aerators, the units can be either fixed-mounted or float-mounted. In fixed-mounted units, the vertical support columns can be constructed of concrete pipe, steel beams, wood piling, and even bridge-mounted when the width is within practical limitations. Fixed-mounted units should have adjustable baseplates or other methods for varying the submergence of the rotor. Very frequently, surface aerators for large aerated waste stabilization basins are float-mounted. This has the advantages of allowing fluctuations in liquid level and ease of repositioning (or removing entirely) the aerators. Also, in most cases, a more economical installed cost may result. (1)

A diffused air system consists of diffusers submerged in the sewage, header pipes, air mains and blowers. Diffused air systems are much more suitable in climates where extended periods of freezing occur.

Layout

The layout of a stabilization pond system is greatly influenced by the shape of the site acquired. A good layout will minimize the quantity of earthwork required and maximize flexibility of operation. The pond layout should include common dike construction, rounded pond corners to prevent collection of floating materials, and multiple cells to minimize short circuiting.

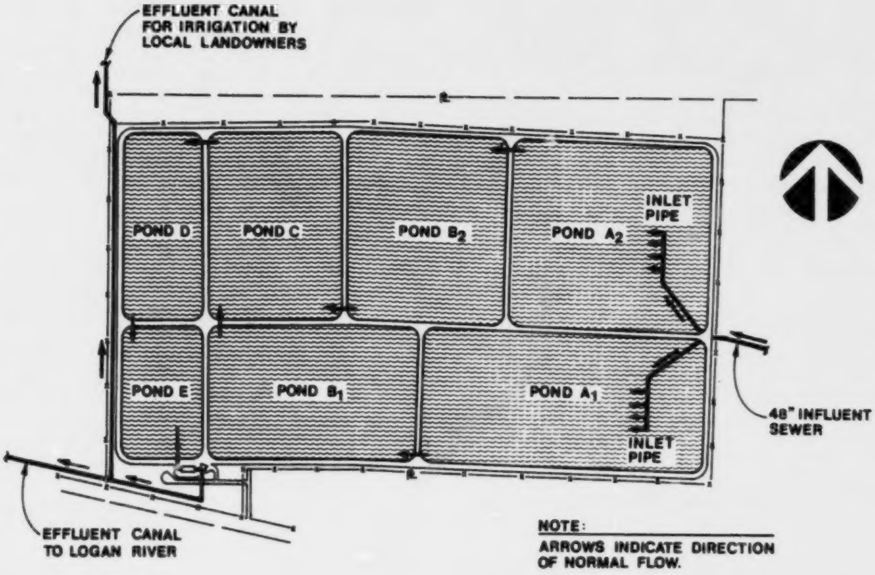
Since the Logan ponds are of substantial size, it was felt necessary and desirable to provide two systems operating in parallel, with some common facilities in the last three cells (Figure 1). The design provides for operating the ponds with any one cell out of service for an extended period of time. Note that a total surface area of 458 acres, and a storage capacity of over 3,000 acre feet is required to serve the City of Logan.

During normal operation, the ponds are designed for a combination of parallel and series operation; i.e., the flow is equally divided at the distribution structure and directed to Ponds A_1 and A_2, which discharge into Ponds B_1 and B_2, respectively. Ponds B_1 and B_2 disdischarge into Pond C and the combined flow continues through Ponds D, E, and the chlorine contact basin. Ponds A_1, A_2, B_1, and B_2 serve to expose a large surface of the raw sewage to both air and sunlight. This exposure promotes growth of algae and other forms of life which utilize nutrients in the sewage for growth. Ponds C, D, and E serve primarily to provide detention for the sewage after treatment in Ponds A_1, A_2, B_1, and B_2. As there is normally no nutrient supplied to these ponds from the raw sewage, the algae population will usually be reduced. The detention time provided by these ponds is also sufficient for a large portion of the coliform bacteria in the sewage to die. Since these ponds were built some 10 years ago, it has generally been found that the coliform requirements of the Utah State Division of Health then in effect could be met without chlorination. The chlorination facilities provided at the Logan ponds have only been used during test periods.

Figures 2 through 5 describe the performance of Logan's stabilization ponds over a 5-year period. The effluent BOD has ranged from 31 mg/l to 1 mg/l with 90 percent of the values less than 18 mg/l. Effluent TSS have been less than or equal to 30 mg/l 90 percent of the time. Extreme values have varied from 100 mg/l to near zero. Ninety percent of the total coliform counts have numbered less than 1000/100 ml. Values as high as 11,500/100 ml and as low as zero have been recorded. The maximum fecal coliform count recorded was 1100/100 ml. Minimum values have reached zero or none detected. Fecal coliform counts have numbered less than or equal to 300/100 ml for 90 percent of the measurements.

Earthwork

The earthwork techniques utilized in constructing a wastewater stabilization pond are generally identical to those used in constructing an earth dam. Vegetation and porous topsoils should be stripped from the pond site. Any sand or gravel pockets should be removed and the subsoil compacted. The pond embankments should also be adequately compacted during construction to provide stability and to prevent seepage. It may be necessary to sprinkle fine-grained materials in order to obtain the optimum


EFFLUENT CANAL FOR IRRIGATION BY LOCAL LANDOWNERS
POND D
POND C
POND B2
POND A2
INLET PIPE
POND E
POND B1
POND A1
INLET PIPE
48" INFLUENT SEWER
EFFLUENT CANAL TO LOGAN RIVER
NOTE:
ARROWS INDICATE DIRECTION OF NORMAL FLOW.

POND	SURFACE AREA (acres)	DEPTH (feet)	VOLUME (acre feet)
A_1	94	6.0	564
A_2	94	6.0	564
B_1	70.5	6.7	472
B_2	70.5	6.7	472
C	63	7.35	463
D	38	7.9	300
E	28	8.5	238
TOTALS	458		3073

Figure 1. Logan wastewater stabilization ponds.

soil moisture content which will yield the maximum soil density.

After the pond dikes have been adequately compacted, they must be trimmed to the specified slope. Exterior slopes should accommodate mowing equipment for ease of maintenance. Interior slopes vary according to the type of soil available and the erosion protection provided. Figure 6 shows the construction of the Logan stabilization ponds and the procedure the contractor used to trim the dikes after placing and compacting the material.

Earthwork design also includes the development of roadways or access ways on top of the dikes. On smaller facilities we have utilized a roadway as narrow as 8 feet, but with ponds as large as those for Logan, greater widths are advisable. The roadways should be graveled to allow access under wet weather conditions.

For the most economical construction, excavation should be designed to balance with fill. Since the Logan pond site consisted largely of 3 to 4 foot hummocks, it was extremely difficult to

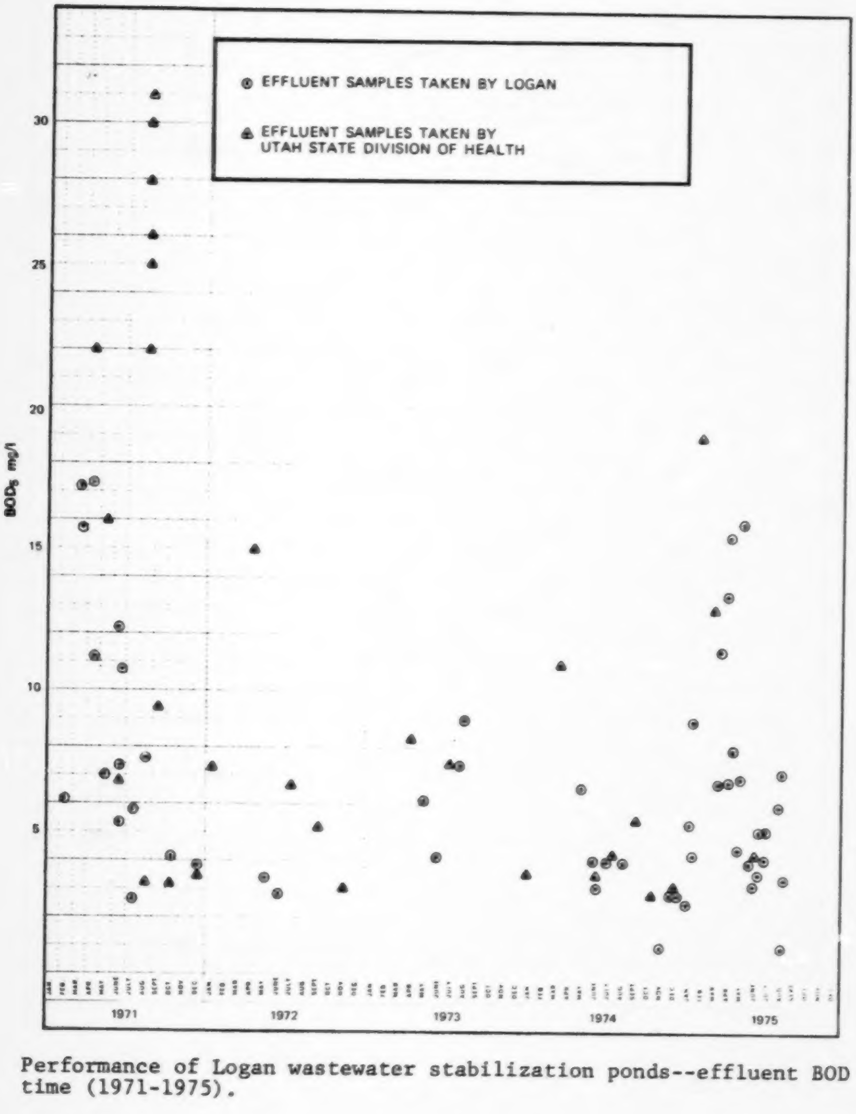


Figure 2. Performance of Logan wastewater stabilization ponds--effluent BOD vs. time (1971-1975).

determine an average or typical elevation. Photogrammetry was used to develop a 50-foot grid of spot elevations. Even with this complete topographical data, obtaining a balance of cut and fill was difficult, since a difference of 0.1 foot elevation was equivalent to about 80,000 cubic yards of material at the Logan site.

We established a construction procedure to permit a balance of cut and fill. It called for stripping the primary and secondary ponds, filling the low spots with the strippings, and then excavating to a predetermined bottom elevation to obtain material for construction of the berm under the influent pipeline and for certain pond dikes. Upon completion of excavation in these cells, adjustments were made in the bottom elevation of the final ponds so that the required excavation quantities could be obtained. This procedure worked very satisfactorily.

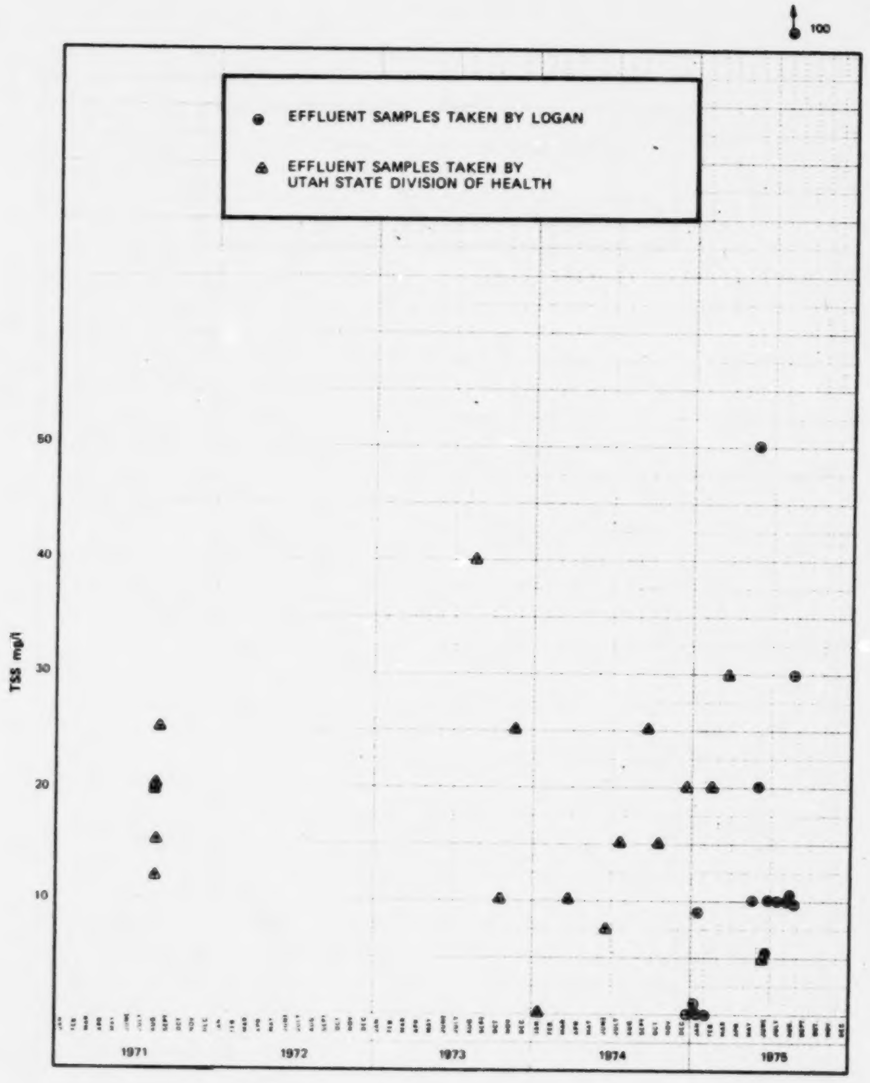


Figure 3. Performance of Logan wastewater stabilization ponds--effluent TSS vs. time (1971-1975).

An additional consideration affecting earthwork is the location of groundwater. At Burley, Idaho, we are involved in upgrading the existing stabilization pond by installing an aerated pond and a pH adjustment cell. Construction of the pH adjustment cell involved excavating to a depth considerably below the groundwater table. In this case, it was necessary for the contractor to dewater the area by well points before he could properly construct the new cell.

Dike Protection and Seepage Control

The protection of earth dikes against wave action and maintenance of a relatively water-tight seal throughout the stabilization pond are major design considerations. Our experience has indicated rather conclusively that dikes forming ponds larger than 10 to 15 acres should be provided with protection from wave erosion. In one of our early projects, a 65-acre facility for the

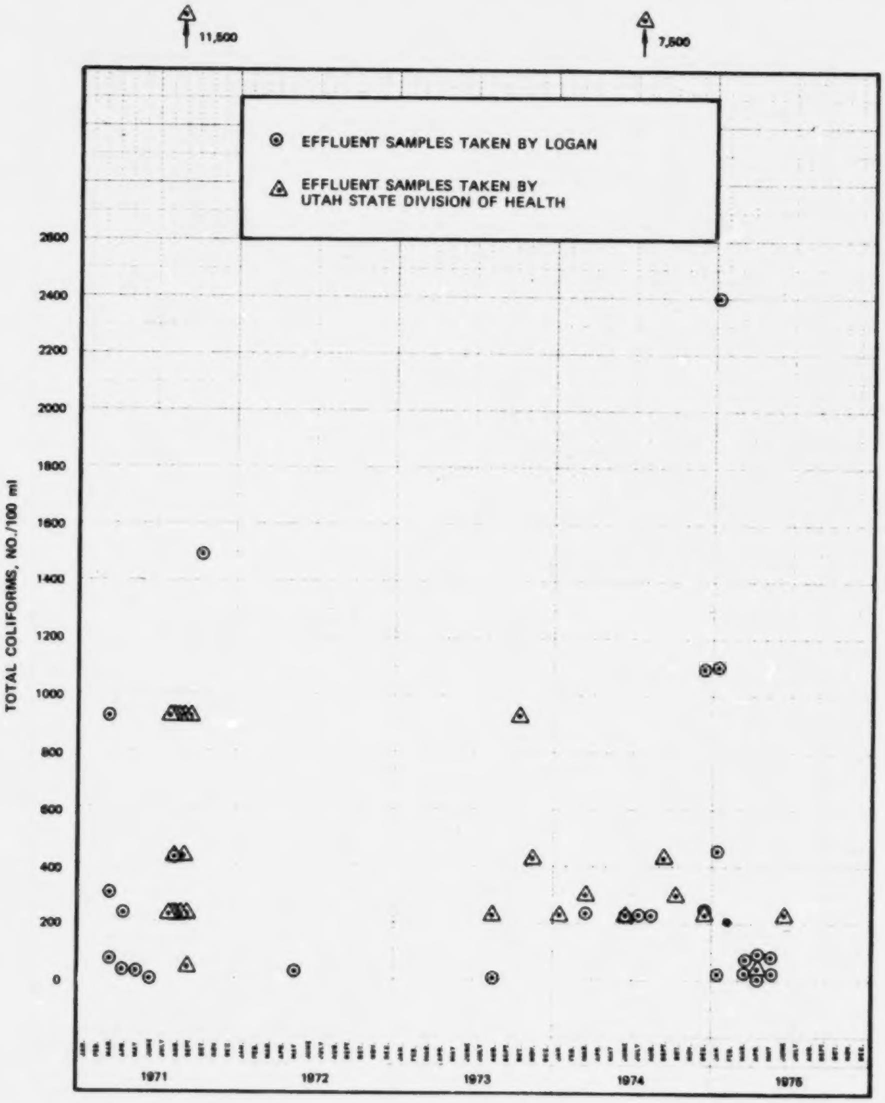


Figure 4. Performance of Logan wastewater stabilization ponds--effluent total coliforms vs. time (1971-1975).

City of Ontario, Oregon, slope protection provided in the initial design consisted only of locally available pitrun gravel, generally under four inches maximum size. During a rather violent thunderstorm in the second year of its operation, wave development was sufficient to cause serious erosion damage to the downwind dikes. Emergency riprapping was required to save the dikes. Riprap has subsequently been provided for the remainder of the pond.

In areas where a prevailing wind exists, it may be permissible to place the heavier riprap on the downwind dikes only; and provide lighter protection on the dikes which are not subject to severe wave action. This can result in considerable cost savings.

Figure 7 is an illustration of a relatively small pond that experienced severe interior dike erosion because proper slope protection was not provided.

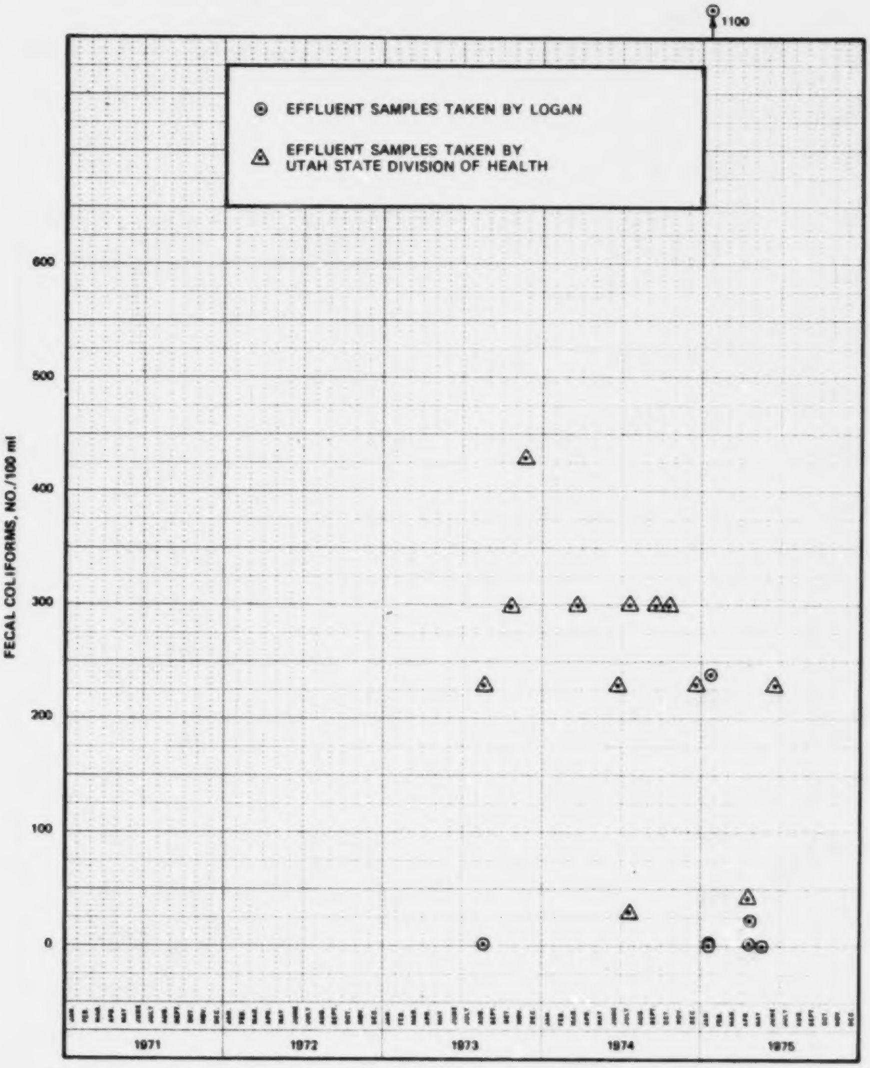


Figure 5. Performance of Logan wastewater stabilization ponds--effluent fecal coliforms vs. time (1971-1975).

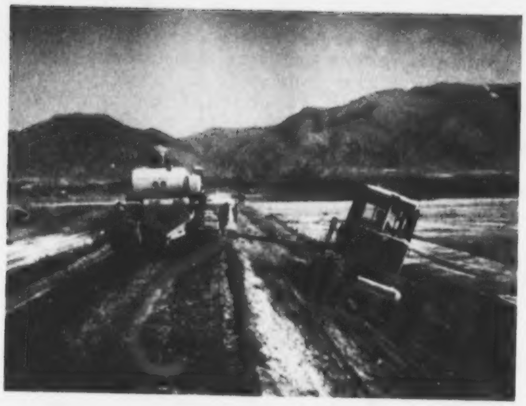


Figure 6. Trimming dikes at the Logan wastewater stabilization ponds.

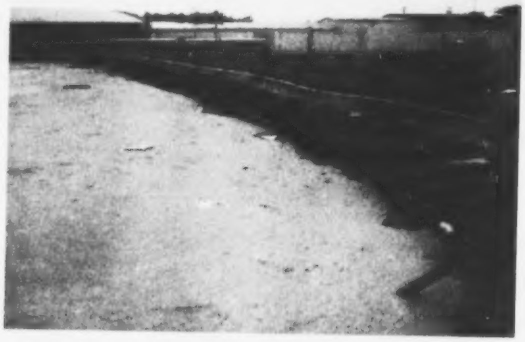


Figure 7. Erosion of interior dike surface at a wastewater stabilization pond in Greenville, Michigan.

In addition to protection of the interior surfaces of the dike, it is necessary to protect the exterior dike surfaces from erosion due to runoff and wind. We frequently place topsoil on exterior slopes and seed the slopes with a suitable variety of native grass. In semiarid areas typical of much of the Intermountain West, dryland grasses such as crested wheat have survived well without irrigation.

In some instances where mechanical aeration is involved, it may be necessary to provide a layer of gravel, or a concrete pad, of sufficient size on the bottom of the pond to prevent scour due to the velocities developed by the aeration equipment.

At Logan, a relatively heavy riprap was used to protect the interior dike slopes. The need for the heavy riprap resulted from the steep interior slopes, potential high wind velocities, and the large "fetch" common in ponds of this size. The slope of the dike directly affects the amount of wave energy absorbed, and the energy of the water scouring the face of the dike. A more gradual slope lessens the effect of the waves. However, flat slopes provide shallow areas ideal for emergent vegetation.

Figure 8 shows a typical section of the dikes constructed at Logan. A 0.8 foot gravel filter was placed on the compacted dike and overlaid with 1 foot of quarried riprap. The exterior surface was seeded with a mixture of 60 percent tall wheat grass and 40 percent crested wheat grass. The dikes are of sufficient height to allow a 3 foot freeboard.

At some installations, it may be necessary to lower the phreatic surface or water table within the exterior dikes to provide embankment stability and to control seepage at the exterior toe. A

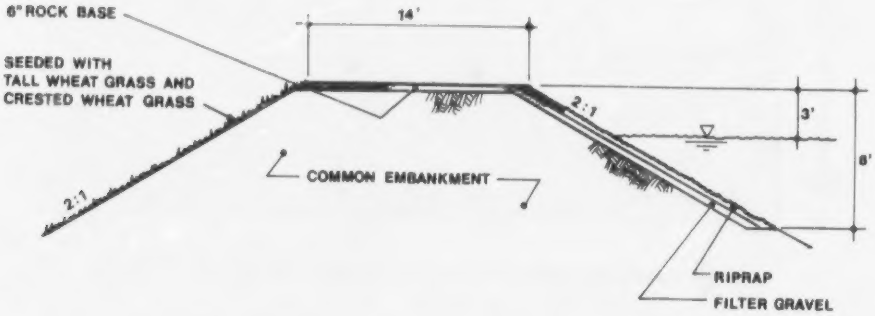


Figure 8. Method of dike protection at the Logan waste stabilization ponds.

filter blanket similar to that shown in Figure 9 is excellent for such conditions.

Seepage control is also required around any pipeline penetrating a dike. Concrete collars have been utilized to prevent erosion and to reduce the likelihood of seepage along the pipeline. Proper installation of pipes passing through dikes can be assured by building up the dike at least 2 feet above the pipe elevation, then cutting a trench for the pipe. The trench should be filled and compacted with an impervious backfill.

Some of the alternatives available for controlling seepage in stabilization pond bottoms are identified in Figure 10. This figure also illustrates the general effectiveness of each approach as compared to the cost. The use of concrete, asphalt, and soil cement is limited to special situations due to their high cost. Soil sealing chemicals and bentonite find limited application due to their low effectiveness. If suitable materials are available, it is our preference to use an earth blanket for sealing the pond bottom. However, the use of any pond liner must be evaluated according to the specific site requirements. If the ponds are to remain empty for a prolonged period, consideration must be given to the possible effects from freezing and thawing during cold weather or cracking from hot, dry weather. Freezing and thawing will generally loosen a soil liner for some depth. We recommend maintaining an adequate volume of water in earth-lined ponds at all times to prevent breakup of the pond bottom. The liquid depth should also be sufficient to prevent weed growth. Roots from weeds can form conduits in the pond bottom.

When suitable materials are not available or suitable conditions do not exist for earth blankets, synthetic membranes such as PVC are commonly used.

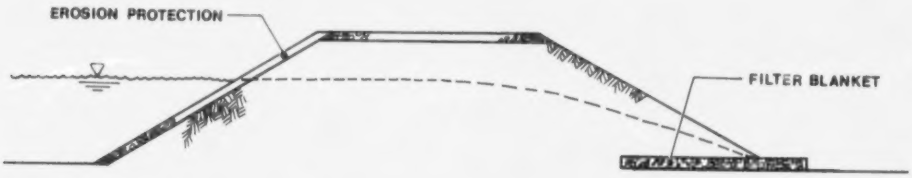


Figure 9. Dike section showing filter blanket installation.

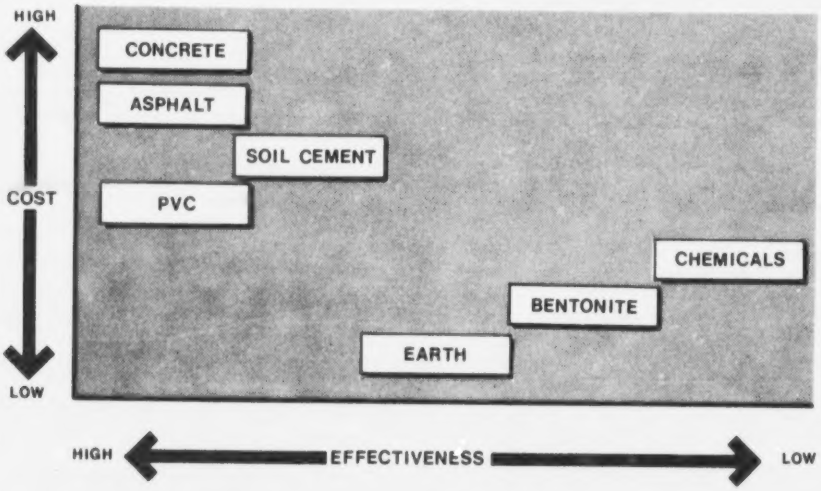


Figure 10. Relative cost/effectiveness of various pond liners.

Although these membranes are effective sealants, special installation procedures are required to prevent damage to the membrane. We recommend the use of a minimum 20 mil thickness to minimize the danger of puncture. It is necessary to support the membrane with a carefully prepared granular material. We also consider it necessary to cover the membrane after installation to hold it in place and provide additional protection. This covering should consist of fine-grained materials which, in turn, may require a covering of well-graded gravel or some other material that is sufficiently heavy to prevent erosion of the fine material that protects the PVC liner.

We have had some unhappy experiences with PVC lining. Early in the history of aerated ponds, some manufacturers of this material felt it was unnecessary to cover the lining after installation. We cannot recommend that practice.

Hydraulics

In smaller ponds, the inlet structure normally consists of a single influent pipe which terminates near the center of the pond. Design conditions at Logan indicated that a more complex structure was necessary. Since these ponds are very large, it was considered essential to utilize a full scale diffuser to distribute the influent over a large area of the primary cells and to minimize short circuiting. Furthermore, Logan experiences extremely high seasonal variations of flow due to summer infiltration of irrigation water. A dual diffuser was selected to handle these variations in flow.

Figure 11 shows a plan view of the Logan diffuser. The inlet was designed so the majority of the wastewater solids and influent flow would always be handled by the main diffuser. This reduced the potential for solids buildup within the diffusers and around the diffuser outlets. When the influent flow exceeds the capacity of the main diffuser, the influent overflows into the secondary diffuser. An inlet pad common to the main and secondary diffusers prevents scouring of the pond bottom. A typical diffuser outlet used at Logan is shown in Figure 12.

Flow measurement is commonly accomplished by the use of a parshall flume. Because the throat width is constant, the discharge can be obtained from a single upstream measurement of depth. Flow measurement allows the operator to control the pond system for optimum treatment of the wastewater.

Structures

Correctly designed inlet, transfer, and outlet structures are the keys to proper hydraulic operation of stabilization ponds. Influent structures should properly distribute the wastewater flow to diffusers or inlet pipes. Pond transfer structures should be valved or provided with other arrangements to regulate flow between ponds and permit variable depth control. Scum baffles and stop-log guides should be provided on structures between ponds that have a potential for undesirable floating debris. The pond effluent structure should be equipped with multiple drawoff lines or some other adjustable drawoff

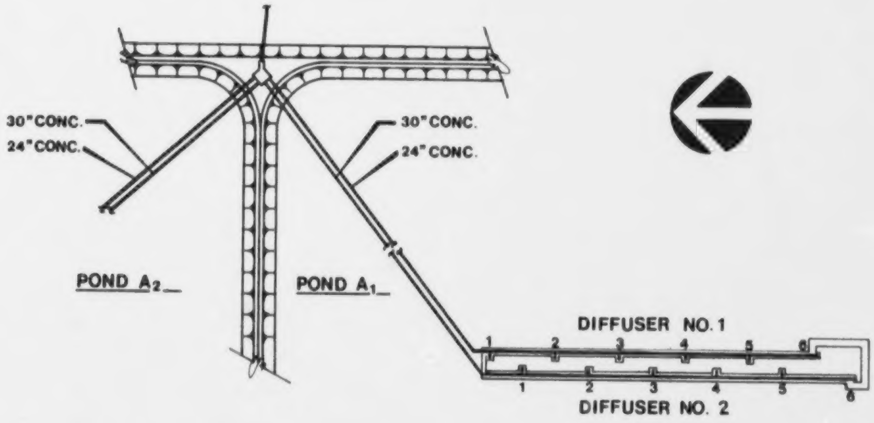


Figure 11. Diffuser and distribution box layout at the Logan wastewater stabilization ponds.

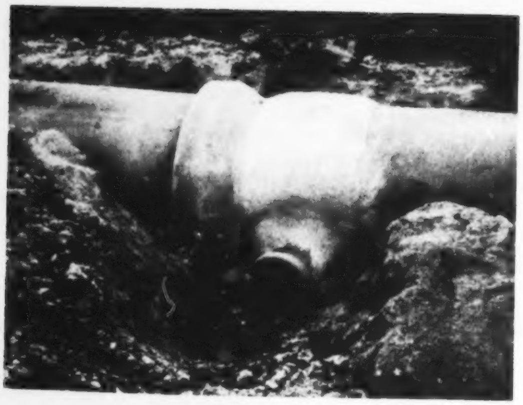


Figure 12. Typical diffuser outlet at Logan ponds.

device, so effluent can be withdrawn from selected depths. Under normal operation the lowest drawoff lines should be high enough above the pond bottom to prevent eroding velocities and avoid picking up bottom deposits. Provisions should also be included to allow for drainage of the pond.

Since a stabilization pond is frequently inviting to trespassers, it is also recommended that a fence and warning signs be placed around the perimeter of the pond advising of the pond contents.

Leakage Test

In our opinion, newly constructed ponds should be subjected to a leakage test, except in rare instances. A leakage test was not conducted at Logan because of the relatively impervious soil conditions, and the artesian aquifer beneath the site. Ponds in areas where surrounding homes draw their water supply from wells should always be tested.

The amount of actual leakage from stabilization ponds may be difficult to determine. At a facility with a large surface area, evaporation becomes very significant and must be taken into account when calculating the pond seepage rate. Our estimate of average evaporation at Logan during summer months was on the order of 1 cubic foot per second. We estimated the average annual evaporation at 0.5 mgd. In our opinion, regulations that specify a seepage rate less than 1/4 inch per day for earth ponds are unrealistic. First, actual evaporation from the pond surface cannot be calculted accurately enough to determine such low seepage rates. Secondly, it is doubtful that earth-liners are capable of providing a sufficiently tight seal to yield seepage rates below 1/4 inch per day, especially under a high head.

CONCLUSION

Our experience with the many types of treatment facilities has convinced us that a properly designed and constructed stabilization pond is a highly reliable and effective form of waste treatment. The key to designing and constructing a successful facility lies in performing the various design elements with the same care that would be utilized on a large earth dam.

REFERENCES

(1) Wastewater Treatment Plant Design, prepared by a joint committee of the ASCE and WPCF, Lancaster Press, Inc., 1977.

A CASE HISTORY EXAMINATION OF LAGOON UPGRADING TECHNIQUES

L. Sheldon Barker*

There are many cities in the southern Idaho, eastern Oregon, and northern Utah areas using stabilization ponds to treat their wastewater. Most of these ponds were built in the 1960s and now need some form of effluent upgrading to meet current Environmental Protection Agency (EPA) requirements for secondary treatment. CH2M HILL has had the opportunity to complete "201" facilities plans for several towns with lagoon facilities including: La Grande, Baker, and Ontario, Oregon; Burley, Idaho; and Logan, Utah. All of these cities have flows exceeding 1 million gallons per day (mgd) which places them in the top 5 percent of communities using lagoon treatment(1).

The EPA has established secondary treatment requirements whereby towns with design flows in excess of 2 mgd must meet the following effluent quality:

1. Biochemical Oxygen Demand (BOD) $\leq$ 30 mg/l.

2. Total Suspended Solids (TSS) $\leq$ 30 mg/l.

3. The treatment facility must provide a minimum removal of 85 percent of the influent BOD and TSS.

Wastewater lagoons, by themselves, will not meet these treatment requirements, principally because of the large suspended solids load contributed by algae growth in the ponds. Lagoons offer advantages, however, that amply justify their continued use, even though algae removal processes must follow.

Lagoon treatment provides:

1. Simple operation almost totally free of operational control requirements.

2. Stable operation that has a low probability of malfunctioning. This stability applies both to the biological processes, where the large volume of a lagoon provides tremendous buffering capacity against organic shock loads, and to mechanical simplicity. Mechanical equipment used with lagoons is limited principally to pump stations and aeration equipment.

3. A very low energy requirement, again due to the general lack of mechanical equipment necessary to operate lagoon.

4. An ability to convert ammonia to nitrate. A well designed lagoon will achieve almost total biological conversion of ammonia to nitrate, a more innocuous form of nitrogen.

When secondary treatment standards were first promulgated for wastewater lagoons, back in the early 1970s, there were few proven methods for polishing lagoon effluent. Only chemical addition (alum) and clarification followed by dual media filtration had at that time been operated successfully, at a 0.5 mgd facility in Lancaster, California (2). The other method then available for meeting discharge requirements was land disposal, using lagoon effluent directly without the need for further treatment. Some of the other alternatives being investigated at that time included: chemical addition followed by dissolved air flotation, mixed media filtration, intermittent sand filtration, anaerobic rock filters, and microstraining.

Even the two proven polishing techniques were not without drawbacks. Chemical addition and clarification or dissolved air flotation requires expensive doses of alum and polymer to coagulate the single cell algae species present in wastewater lagoons. In addition, this alternative requires operator sophistication and attention. Land treatment is a relatively simple technical alternative, but is often administratively difficult to implement. Land treatment requires large acreages of contiguous property which can be difficult to obtain, as many existing property owners may be involved. Irrigation districts may be persuaded to accept lagoon effluent but very often the farm community is reluctant to accept the crop restrictions that many states place on the use of this water.

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More recently, treatment alternatives are under development which show promise of upgrading lagoons while maintaining their traditional advantages of simplicity and stability. Microstraining is an example of recent technological advancement. Polyester fabric has been produced reducing the sieve size available to 1 micron, much below the 23 micron stainless steel screens that were previously available.

The evolution of lagoon upgrading technology is perhaps best understood by examining case histories of cities and the treatment alternatives which they tested and selected. The three case histories of lagoon upgrading presented in this paper are:

1. Ontario, Oregon--Pilot testing of phase isolation.

2. La Grande, Oregon--Selection of air flotation and filtration.

3. Burley, Idaho--Pilot testing of microscreening and aquaculture.

ONTARIO, OREGON

The City of Ontario has a four pond, series lagoon system with pond sizes of 45 acres, 23 acres, 9.5 acres, and 9.0 acres followed by chlorination.

The system has an average annual flow of about 1.7 mgd and must be upgraded to meet secondary standards. Figures 1 and 2 show various pond treatment efficiencies. Note that the effluent violates TSS requirements.

The facilities plan (3) originally proposed chemical addition and sedimentation as the cost effective treatment alternative, after jar tests indicated alum would be an effective algae coagulant. The city's primary concern, however, was to minimize operation and maintenance costs. They decided to investigate further a less proven treatment alternative: phase isolation. With assistance from CH2M HILL, Ontario removed Pond No. 3 (see Figure 3) from their lagoon treatment scheme and set up a phase isolation pilot test program modeled after the successful Woodland, California, operation (4). Initially the program was delayed while excessive leakage in Pond 3 was corrected using betonite. The phase isolation pilot test has been running since March 1977. Results of the program are shown in Table 1. The middle column of Table 1 indicates the minimum holding time in the isolation pond required to produce a secondary treatment quality of 30 mg/l BOD and TSS. This holding time is important as the primary design factor for a phase isolation system. Since testing was not done daily, the residence times are probably somewhat higher than the absolute minimum required. Two conclusions can be drawn from these data:

1. The minimum isolation times varied from run to run without an easily

Table 1. Ontario, Oregon, phase isolation program.

Run No.	Approx. Date	Earliest Isolation Time To Meet Secondary Treatment	Lagoon Quality Obtained at the Specified Isolation Time	
			BOD_5	TSS
1	March 1977	7 days	20 mg/l	18 mg/l
2	August 1977	28 days	18 mg/l	27 mg/l
3	September 1977	21 days	5 mg/l	19 mg/l
4	December 1977	28 days	18 mg/l	16 mg/l
5	January 1978	a	44 mg/l	40 mg/l
6	February 1978	12 days	27 mg/l	30 mg/l
7	March 1978	17 days	16 mg/l	17 mg/l
8	June 1978	b	21 mg/l	34 mg/l

[a] Pond No. 3 did not meet secondary treatment during a 28-day holding period.
[b] Pond No. 3 did not meet secondary treatment during a 40-day holding period.

discernible pattern. Selecting an isolation time for use as a design parameter would be difficult.

2. Secondary treatment was not achieved on 25 percent of the runs, even after isolation periods of up to 40 days.

The winter of 1977 had above normal temperatures in eastern Oregon so the pond never developed its usual extensive ice cover. Operating personnel reported that the ice which did occur minimized the treatment capability of phase isolation.

Phase isolation does not seem able to produce a satisfactory level of treatment for Ontario, especially since the State of Oregon, Department of Environmental Quality (DEQ) water quality guidelines could eventually require a summertime BOD and TSS effluent of 20 mg/l. Phase isolation may, however, provide a benefit to those smaller cities which have been granted a suspended solids waiver and, therefore, are not required to completely meet secondary treatment standards.

LA GRANDE, OREGON

The City of La Grande has a two cell lagoon system with a total surface area of 100 acres. Selection of the cost effective method to upgrade treatment for

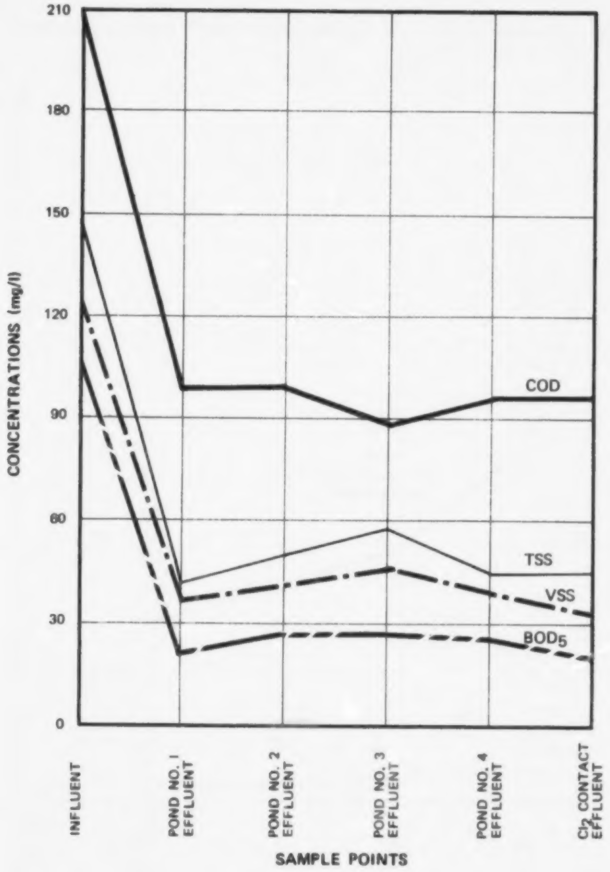


Figure 1. Variations in COD, TSS, VSS, and BOD_5 through the wastewater lagoon system, 18-20 November 1974.

this community of 11,000 was complicated by several factors:

1. The existing discharge into Gekeler Slough (a tributary of the Grande Ronde River) cannot continue because there is no dilution water available during the summer and fall. Ice blockage has also caused downstream flooding during the winter.

2. The Grande Ronde River, the alternate discharge point, offers more flow, but is almost 5 miles from the existing lagoon site. Even this receiving stream has low summer flows and the DEQ is requiring the following dilution restriction:

$$\frac{(\text{BOD}_5)(\text{Effluent Flow})}{(\text{Receiving Stream Flow})} \le 1$$

3. The city sewer system is old and suffers from infiltration, especially during winter months. Not all the infiltration is cost effective to remove, thus the 85 percent secondary treatment requirement often requires an effluent quality substantially below 30 mg/l.

Table 2 predicts the cumulative discharge requirement that La Grande will have to meet at the Grande Ronde River. The dilution governing effluent criteria in Table 2 were based on average summer water flows in the Grande Ronde River and low water years will impose even more stringent effluent requirements. The upgrading alternative must be capable, therefore, of producing a 10 mg/l effluent quality for BOD and TSS. This severely restricted the alternatives available.

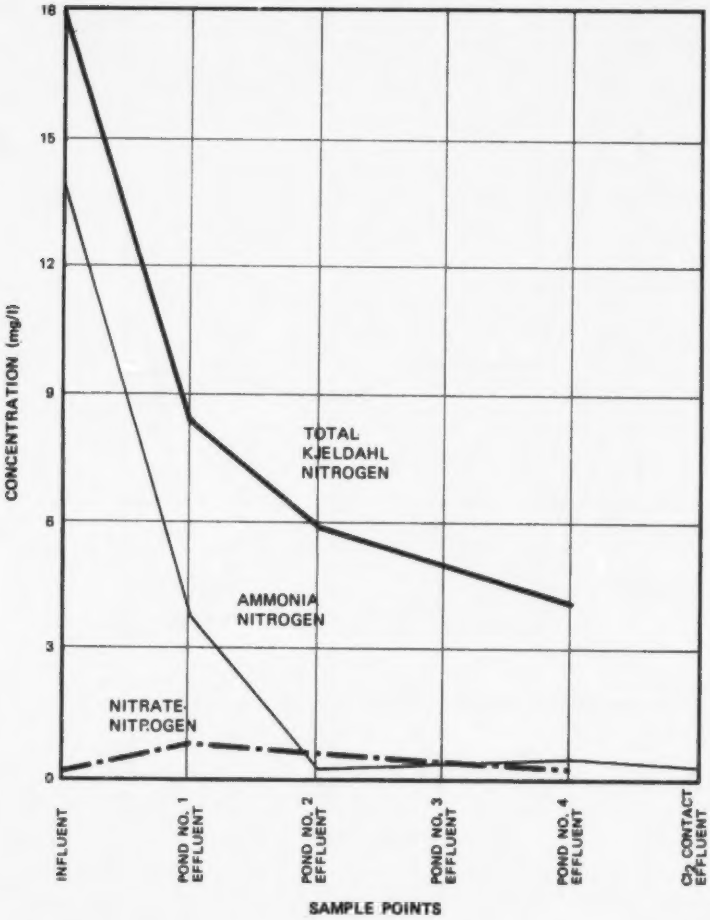
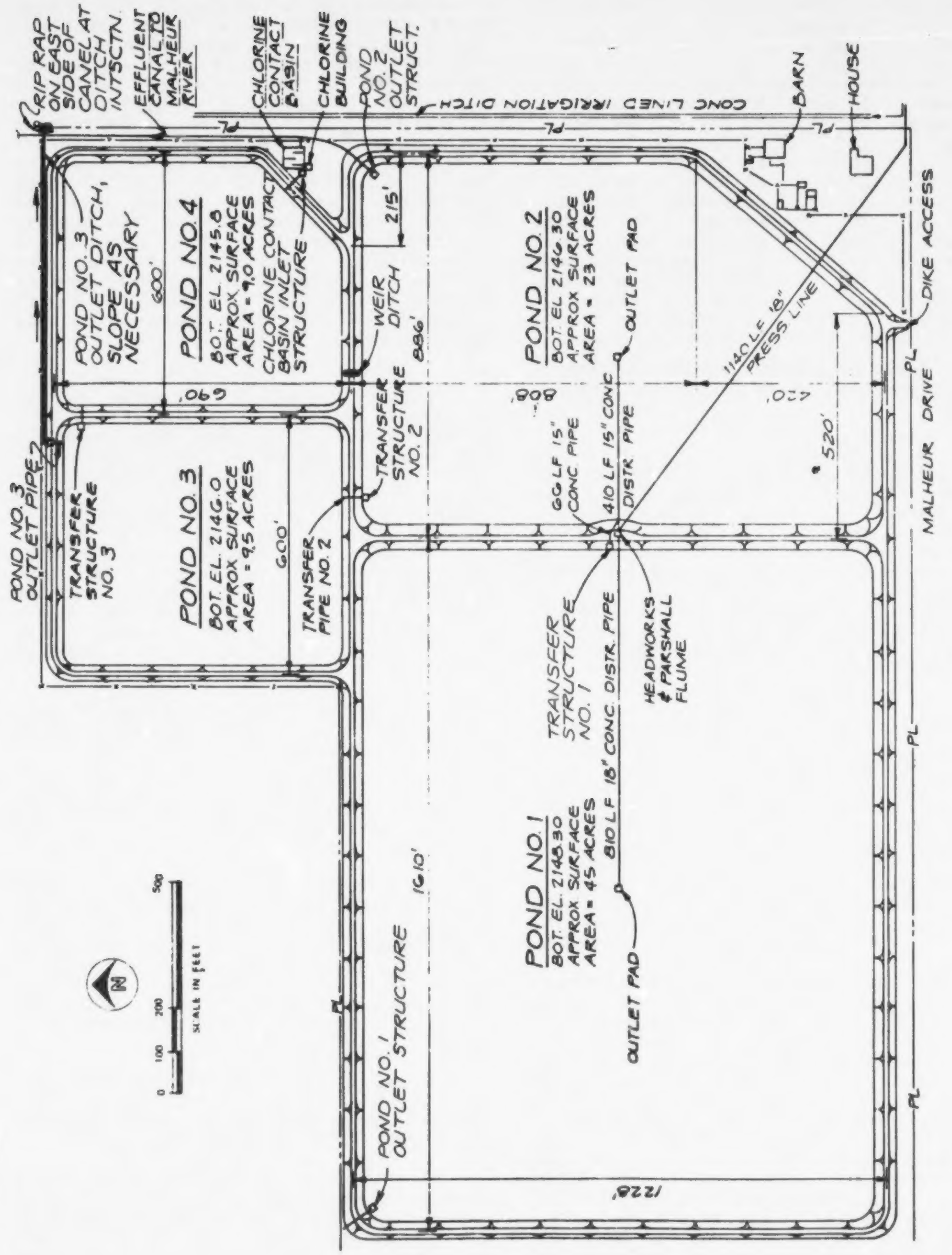


Figure 2. Nitrogen balance through the wastewater lagoons, 18-20 November 1974.



POND NO.1
BOT. EL. 2148.30
APPROX SURFACE
AREA = 45 ACRES
POND NO.2
BOT. EL 2146.30
APPROX SURFACE
AREA = 23 ACRES
POND NO.3
BOT. EL. 2146.0
APPROX SURFACE
AREA = 9.5 ACRES
POND NO.4
BOT. EL. 2145.8
APPROX SURFACE
AREA = 9.0 ACRES
POND NO.1 OUTLET STRUCTURE
POND NO.3 OUTLET PIPE
POND NO.3 OUTLET DITCH, SLOPE AS NECESSARY
POND NO.2 OUTLET STRUCT.
TRANSFER STRUCTURE NO. 1
TRANSFER STRUCTURE NO. 2
TRANSFER STRUCTURE NO. 3
TRANSFER PIPE NO. 2
810 LF 18" CONC. DISTR. PIPE
410 LF 15" CONC DISTR. PIPE
60 LF 15" CONC PIPE
1140 LF 18" PRESS LINE
OUTLET PAD
OUTLET PAD
HEADWORKS & PARSHALL FLUME
CHLORINE CONTACT BASIN INLET STRUCTURE
CHLORINE CONTACT BASIN
CHLORINE BUILDING
WEIR DITCH
RIP RAP ON EAST SIDE OF CANEL AT DITCH INTSCTN.
EFFLUENT CANAL TO MALHEUR RIVER
CONC LINED IRRIGATION DITCH
BARN
HOUSE
DIKE ACCESS
MALHEUR DRIVE
PL
1610'
1228'
600'
600'
690'
808'
886'
420'
520'
215'
0 100 200 500
SCALE IN FEET
N

Figure 3. Phase isolation pilot study-site plan.

Table 2. La Grande, Oregon, required effluent quality.

Month	Maximum BOD & TSS	Limiting Factor
January	20 mg/l	85 Percent Removal
February	17	85 Percent Removal
March	18	85 Percent Removal
April	20	85 Percent Removal
May	27	85 Percent Removal
June	27	85 Percent Removal
July	30	Secondary Treatment
August	14[a]	Dilution Requirement
September	16[a]	Dilution Requirement
October	12[a]	Dilution Requirement
November	30	Secondary Treatment
December	30	Secondary Treatment

[a] Based on summer flow in the Grande Ronde River during average water years.

Jar tests with alum were run on the lagoon effluent. Although the algae flocculated very well, it maintained a neutral buoyancy. The floc appeared to be buoyed by small trapped gas bubbles, even though the jar tests were performed in the dark to eliminate continued O_2 production. A hypothesized explanation is that the algae had supersaturated the pond with oxygen and that this oxygen came out of solution in the rapid mix portion of the jar test. Because of the buoyancy condition, the feasibility of using clarification, such as was done in Lancaster, California, did not appear favorable. Air flotation was the superior choice. Filtration was added following the air flotation to polish the effluent and produce the required treatment. A moving bridge backwash filter (such as manufactured by Environmental Elements Corporation) was selected for its low head requirement. This eliminated the need for an intermediate pump station.

The facilities plan (5) indicated that upgrading the lagoon in the manner shown in Figure 4 was the cost effective solution. Design criteria developed for this alternative are shown in Table 3. Construction bonds for the existing lagoon have almost been paid for and the facility does not warrant abandoning in favor of an activated sludge facility. Land application of lagoon effluent was examined closely since the Grande Ronde valley is a strong agricultural area. Freezing winter conditions and high spring groundwater severely limited the application season, and ultimately led to a prohibitively high alternative cost.

Even after the facilities plan had been adapted, the city remained quite concerned about the necessity of discharging directly to the Grande Ronde River with an effluent pipeline to be built at an estimated cost of $650,000. A new alternative was developed using the existing discharge at Gekeler Slough as a receiving stream during the winter and as an irrigation canal during the summer. To use Gekeler Slough as an irrigation canal the city had to guarantee that there would be no discharge from Gekeler Slough into the Grande Ronde basin during the summer irrigation season. This alternative also included a small fall storage pond to handle effluent during fall months (after the irrigation season) until adequate dilution flows become available again in Gekeler Slough.

Summer irrigation discharge was an attractive alternative to the city. Construction of the effluent pipeline would not be necessary and a lower quality effluent would be allowed during the summer months, thus reducing operation costs for power and chemicals. Implementation of this alternative required that an irrigation district be formed to distribute water among the area farmers. The district would contractually agree with the city to handle all their summer flows over the next 20 years. Oregon water right laws complicated the formation of the district because:

1. Water rights existed on the lagoon effluent which was then being used for irrigation.

2. The oldest water rights entitled its holders to use lagoon effluent, whether they belonged to the district or not.

3. Farmers without water rights were afraid that if they joined the district, while those with older water rights did not, the only time they would get any water would be in wet years. During those times they would be forced to take water they didn't need, and their crops would be damaged as a result.

Legal evaluation of the district formation required, therefore, that participation by all 13 area farmers would have to be unanimous.

The City of La Grande tried for 9 months to help get the irrigation district formed. Several public meetings were held and three different draft agreements between the city and the irrigation district were prepared. Even in the drought year of 1977, the farm-

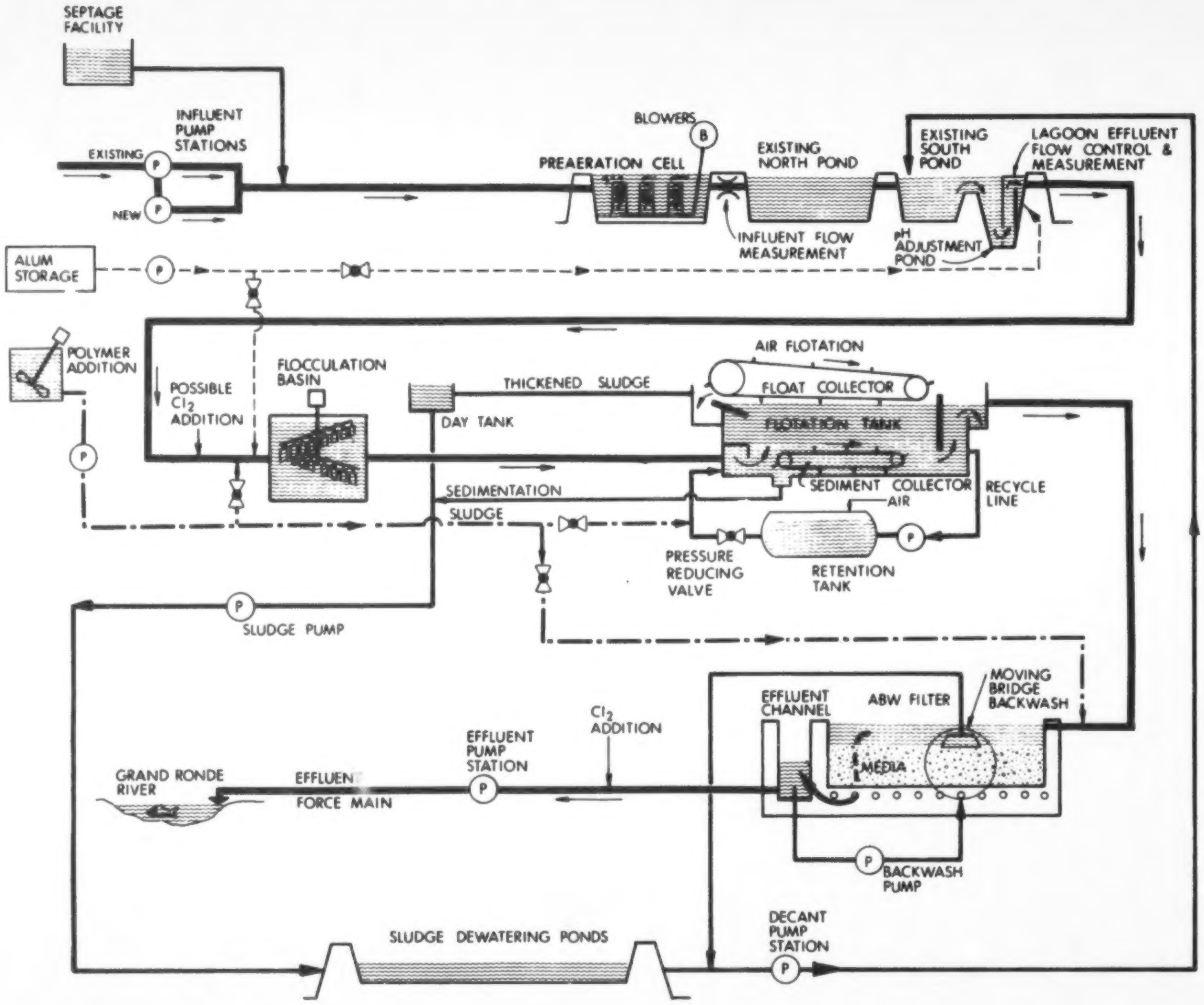

SEPTAGE FACILITY
INFLUENT PUMP STATIONS
EXISTING
NEW
P
BLOWERS
B
PREAERATION CELL
EXISTING NORTH POND
INFLUENT FLOW MEASUREMENT
EXISTING SOUTH POND
LAGOON EFFLUENT FLOW CONTROL & MEASUREMENT
pH ADJUSTMENT POND
ALUM STORAGE
POLYMER ADDITION
POSSIBLE Cl2 ADDITION
FLOCCULATION BASIN
DAY TANK
THICKENED SLUDGE
AIR FLOTATION
FLOAT COLLECTOR
FLOTATION TANK
SEDIMENT COLLECTOR
AIR
RECYCLE LINE
SEDIMENTATION SLUDGE
PRESSURE REDUCING VALVE
RETENTION TANK
SLUDGE PUMP
EFFLUENT CHANNEL
ABW FILTER
MOVING BRIDGE BACKWASH
MEDIA
Cl2 ADDITION
EFFLUENT PUMP STATION
GRAND RONDE RIVER
EFFLUENT FORCE MAIN
BACKWASH PUMP
SLUDGE DEWATERING PONDS
DECANT PUMP STATION

Figure 4. Schematic diagram: lagoons/chemical treatment/filtration.

ers never agreed to the district formation, principally because:

1. There was still concern over the winter flood potential in Gekeler Slough.

2. The DEQ required the farmers to limit to non-human consumption the types of crops they could grow.

3. Water is still relatively plentiful in the Grande Ronde basin and the farmers preferred to be independent.

The city decided that the only viable alternative was to build the pipeline to the Grande Ronde River. Ironically a portion of this pipeline crosses a farm whose owner is requiring up to 500 gpm of summer flow (for his irrigation) as a condition of his property easement. The city is in the process now of checking other farms along the pipeline right-of-way to see if there is further interest in using this water for irrigation. The advantage in this case is that neither the farmers nor the city have to sign a long term contract.

BURLEY, IDAHO

Burley is a community in southeastern Idaho with a population of approximately 10,000. The city has a two cell series lagoon system, with a 36-acre primary cell and a secondary cell of 44 acres. Phase I improvements to the pond are presently underway including construction of a 4.4 million gallon aeration pond (to precede the stabilization lagoons and increase system capacity) and chlorination facilities. Static tube aerators were selected for the aeration pond over surface aerators because: 1) Floating aerators are difficult to operate under winter icing conditions; 2) diffused air systems should not lower the winter water temperature as rapidly as floating aerators. Since biological activity is a function of temperature, the difference in cooling rates relates directly to effluent quality.

The Phase I improvements are not expected to satisfy effluent requirements. The project was broken into two phases while the city waited for a response to its request for a suspended

Table 3. Upgrade design factors La Grande wastewater treatment facilities.

Item	Facilities Required
I. AERATION CELL	
A. Blower Facility	
Number of Units	
(Now)	3
(Future)	1
Type	Centrifugal
Capacity, Each	1,250 scfm
B. Aeration Equipment	
Number of Units	165
Type	Helixical Mixing
Av. Capacity, Each	22 cfm
C. Aeration Cell	
Liquid Depth (min.)	11.5 feet
Detention Time @ Maximum Month Flow	3 days
Detention Time, Peak Flow	1.5 days
II. CHEMICAL TREATMENT	
A. Flocculation Basin	
Number of Flocculators	1
Energy Input, G	40 sec^{-1}
Basin Detention Time, Peak Flow	18 Minutes
B. Air Flotation	
Number of Units	2
Overflow Rate (including recycle @ peak flow)	1.8 gpm/feet^2
Recycle Ratio, Minimum	16 Percent
Air to Solids Ratio	0.015
Operating Pressure	80 psig
C. Sludge Pumps	
Number of Pumps	2
Type	Positive Displacement
Capacity Each	70 gpm
III. FILTRATION	
Number of Units	1
Type	Automatic Backwash (ABW)
Overflow Rate Maximum	3 gpm/ft^2
Backwash Rate	20 gpm/ft^2
IV. SLUDGE LAGOONS	
Number of Cells	4
Liquid Depth	2 feet
Area, Each Cell	2 acres
Ultimate Disposal Method	Landfill

solids waiver. The State of Idaho, Department of Health and Welfare, has recently determined that Burley must provide secondary treatment. Because of high system infiltration, the secondary treatment requirement for 85 percent removal translates to an effluent standard of 20 mg/l for BOD and TSS.

The Burley Facilities Plan (6) and subsequent addendum indicated the cost effective Phase II alternative would be chemical addition (alum) followed by air flotation and dual media filtration. Even though this alternative was cost effective, it was also recognized to be operationally cost intensive. Recent advances in the science of wastewater treatment offer the potential for a lower cost innovative treatment system using the existing Burley lagoons and adding both the following two components:

1. Microstraining--recently developed polyester fabrics for microstrainers have reduced the effective pore sizes to the range of 1 micron. Algae in the Burley lagoon have been found to range in the 4 to 5 micron size. Microstrainers should, therefore, be able to remove algae solids without the addition of coagulating chemicals. The algae removed during this process is intended to be recycled back to the primary pond to provide a seed to help maintain optimum algae concentrations. With all algae solids being recycled the system becomes essentially a closed loop and additional solids handling capacity is needed.

2. Aquaculture--Raising a polyculture of fish species, designed specifically to feed on algae, should help minimize the algae solids buildup produced by the microstrainer. The fish will only be present during summer months. Since the fish should gain about one pound for every four pounds of algae they consume, their harvesting should allow sufficient removal of biomass from the lagoons to maintain long term system equilibrium.

Because the process components of this treatment system are relatively untried (especially in colder climates, as in Burley) they must be pilot tested prior to full scale operation.

Zurn Industries, Inc., has volunteered the use of a 4-foot diameter by 2-foot long microscreen to the City of Burley for a 3 month period in the fall of 1978. The pilot microscreen will be operated continuously over this period to determine: 1) The feasibility of using microstrainers for lagoon algae removal in both summer and winter condition; 2) the allowable design loading rate in gallons per minute per square foot of screen; 3) the potential for long term slime buildup on the surface of the microscreen and any resulting deleterious impacts.

The aquaculture testing will be performed under the guidance of Professor Jack Griffith of Idaho State University. The pilot work he will conduct in 1978 includes toxicity and growth rate testing of several species of fish selected either for their ability to directly consume algae or to stimulate zooplankton, which feed on algae. The treatment capability achievable with the aquaculture system will not be determined until the full scale system is implemented, perhaps in 1980. Fish yields on the order of 500 pounds per acre per year are expected.

There are several ways that aquaculture could complement the microscreen operations, including: 1) Aquaculture may allow the microscreens to produce a better effluent than they could alone. 2) By keeping the lagoon in a solids (algae) equilibrium, aquaculture may delay the time when sludge buildup requires that the lagoon be cleaned out. 3) On an annual basis, aquaculture should decrease the algae loading to the microscreen. This decreased loading may prolong the life of the screen fabric. 4) Aquaculture alone may be able to meet effluent quality requirements during summer months, resulting in power savings from not running the microstrainer.

Aquaculture is not developed in Idaho and, therefore, a market for the fish must be found. There is a mink farm near Burley that will pay $35 per ton for fish. Much of the pilot program and early implementation efforts will be directed toward developing a better market for fish. The fish are not intended for human consumption. Income from the sale of fish is not expected to cover the total cost of the aquaculture program, especially during the first years of implementation.

DISCUSSION

Figure 5 shows the results of our literature review and field investigations of algae solids removal techniques. At any required effluent quality, process selection should be based on both a cost effective analysis and on the requirement for maximizing the simplicity inherent in lagoon operation. For example, both microstraining and air flotation can meet a 30 mg/l TSS requirement. Cost considerations aside, microstraining is a superior choice because microstrainers are operated by simple hydraulic considerations and do not require chemical handling.

All alternatives must be evaluated on their potential for implementation as well as their technical and cost effective merits. For example, selecting a land disposal alternative will be fruitless if it will be next to impossible to acquire the necessary land. Regulatory agencies can impose scheduling deadlines which don't allow the time necessary to obtain complicated pipeline rights-of-way, form irrigation districts, or develop long term land lease agreements. These institutional considerations must be considered during alternative selection.

In conclusion, wastewater lagoons provide stable, reliable treatment that unfortunately doesn't quite meet secondary standards. Increasingly, methods are becoming available to upgrade lagoon effluent quality and still retain the numerous advantages offered by stabilization ponds.

ACKNOWLEDGMENTS

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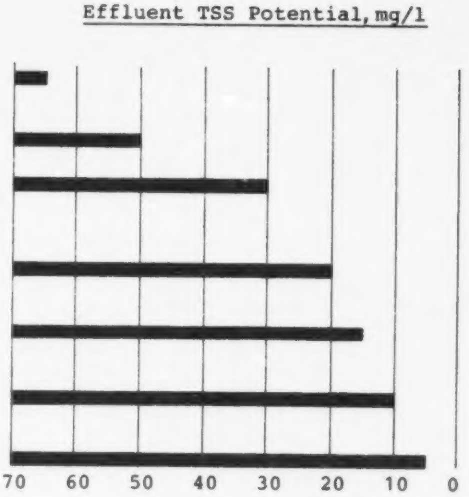
Removal Technique	References
Stabilization Pond Alone	(3) (5) (6)
Phase Isolation	(4)
Air Flotation with Alum Addition	(2) (7) (8) (9) (10) (11) (12) (13) (14) (15)
Microstrainer and Aquaculture*	(8) (16) (17) (18) (19)
Intermittent Sand** Filtration	(8) (20) (21) (22) (23) (24)
Air Flotation & Filtration with Alum Addition	(2) (7) (9) (10) (11) (15)
Land Application	(25) (26) (27) (28)



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*Effluent quality to be determined by pilot testing in Fall 1978. Estimate based on data supplied by ENVIREX.

**Effluent quality based in part on interview with plant operator and site visit at Mt. Shasta, California in March 1977.

Figure 5. Algae removal techniques.

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FIELD EVALUATION OF ROCK FILTERS FOR REMOVAL OF ALGAE FROM LAGOON EFFLUENTS

Kenneth J. Williamson and Gregory R. Swanson*

INTRODUCTION

Aerobic stabilization lagoons are most commonly employed by small municipalities and isolated industrial plants for wastewater treatment. Their popularity in these applications is due to relatively low construction costs and high reliability, and especially due to minimal operation and maintenance requirements. A major limitation of lagoons, however, is the presence of occasionally large quantities of algae in the effluent.

The Clean Water Act of 1972 (Public Law 92-500) required that all municipal effluents meet secondary treatment standards, defined by the EPA as a maximum of 30 mg/l of both 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) on a monthly average basis. Compilation of data on lagoon treatment throughout the U.S., however, soon revealed that most lagoons could not meet a 30 mg/l TSS standard year-round because of the algal content of their effluents.

Consequently, a significant research effort was directed at finding effective methods for upgrading lagoon effluents, especially through the removal of algae. Middlebrooks et al. (1) summarized the results of this research effort and compiled a list of 14 possible techniques.

1. Centrifugation
2. Microstraining
3. Coagulation-Flocculation
4. In-pond removal of particulate matter
5. Complete containment
6. Biological disks, baffles, and raceways
7. In-pond chemical precipitation of suspended materials
8. Autoflocculation
9. Biological harvesting
10. Oxidation ditches
11. Soil mantle disposal
12. Dissolved air flotation
13. Granular media filtration
14. Intermittent sand filtration

In evaluating these processes, emphasis was appropriately placed on the criteria of ease of operation, minimum maintenance and cost, and dependability of operation. Only microstraining, soil mantle disposal, granular media filtration and intermittent sand filtration were considered promising based on these criteria.

An additional, promising alternative for the removal of algae from lagoon effluents is the rock filter. Very simply, a rock filter consists of a submerged bed of rocks (5 to 20 cm diameter) through which the lagoon effluent is passed vertically or horizontally, allowing the algae to become attached to the rock surface and thereby be removed. The basic simplicity of operation and maintenance are the key advantages of this process. The effluent quality achievable and the dependability of long-term operation, however, have not yet been proven.

Beginning in 1970, initial research into the rock filter was undertaken at the University of Kansas (2-6) from which O'Brien (16) concluded that:

1. Peak efficiency for submerged rock filters is achieved during the summer and early fall: this efficiency can produce an effluent meeting 30 mg/l BOD_5-30 mg/l total suspended solids discharge standards. During the winter and spring the efficiency of suspended solids removal decreases significantly.

2. The filters may be anaerobic during the summer and early fall and will produce hydrogen sulfide if sulfates are present.

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3. The rate of solids accumulation in submerged rock filters should allow an effective filter life of greater than 20 years.

4. Rock sizes should be greater than 2.54 cm and less than 12.70 cm.

O'Brien did not elaborate on the basic mechanism of algal removal in rock filters, nor the possible effluent quality which could be achieved at lower loading rates than those used of 0.40-2.67 m3/m3-day (3-20 gpd/ft3).

Recently the EPA has promulgated regulations allowing variances from the secondary treatment TSS limitation of 30 mg/l for municipalities which employ lagoons due to the lack of an appropriate technology to upgrade lagoon effluents. Nevertheless, emphasis on improving effluent water quality necessitates development of dependable lagoon effluent polishing techniques.

The Veneta Rock Filter

A full-scale rock filter was designed and constructed as part of a lagoon expansion and upgrading project at Veneta, Oregon, in 1975. The regional EPA was in full support of this project and contracted with the writers to provide a field evaluation of the rock filter system.

A schematic flow diagram of the Veneta, Oregon, wastewater treatment system is shown in Figure 1. The system treats wastewater from a population of approximately 2200 with no industrial wastewater contribution. Raw wastewater is pumped into the larger first cell and flows through the smaller second cell by gravity. Both lagoons are designed for a minimum water elevation of 0.76 m (2.5 ft) and a maximum of 1.83 m (6.0 ft). Lagoon effluent is then pumped to the rock filter by a submersible pump with a maximum capacity of approximately 25 l/s (400 gpm).

A plan and cross-section of the rock filter is shown in Figure 2 The pressure pipe carrying lagoon effluent discharges into a 0.3 m (1.0 ft) square influent channel upon entering the rock filter. The influent channel is covered with trench grating which improves the distribution of flow. The lagoon effluent rises from the influent channel and moves horizontally towards the discharge weirs where it flows into a covered effluent channel. Finally, the flow from each side of the rock filter is combined, chlorinated, and discharged to the nearby Long Tom River. The rock surface is approximately 0.30 m (1.0 ft) above the water elevation to prevent growth of algae on the rock filter.

The effective surface area of the rock filter is 5400 m^2 (58,000 ft^2) and the effective volume is 8,200 m^3 (290,000 ft^3). The in situ averge porosity of the 7.6 - 15.2 cm (3 - 6 in) rock bed was measured as 42 percent.

REMOVAL MECHANISM

Sedimentation is probably the primary entrapment mechanism in the rock filter due to the large pore sizes. This hypothesis is supported by the observation that algal residues are primarily found on the top of rocks in the Veneta rock filter. As a result, the settling rate of algae is probably the basic physical parameter controlling the efficiency of algae removal.

Measurement of Algal Settling Rates

Algae are typically assumed to settle as individual, non-flocculating particles; this process is classified as discrete settling. Discrete settling is assumed for lakes and ponds (7) and this assumption is extended to include rock filters.

The settling rates of algal cultures can be measured in a settling chamber under quiescent conditions (8, 9, 10). A fluorometer is used to measure the fluorescence of the algal cells as they settle past an optical window. The main advantages of this method are high sensitivity and simplicity. Assumptions and techniques used are described in detail by Stutz and Williamson (9).

Stutz and Williamson (9) found that the variables of temperature and algal species influenced settling rates significantly. Settling rates were not sensitive to either aerobic or anaerobic dark incubation; these would be the conditions found in the rock filter.

Algal settling curves will typically have the shape shown in Figure 3a; an actual curve for a sample from the Veneta lagoon is shown in Figure 3b. This curve represents the recorder traces from the fluorometer output during the settling test and is specific for the fixed settling depth of the settling chamber (usually 10 mm). The exact shape of the curve is determined by the size distribution of the algal culture (8).

The characteristic linear portion of the curve is used to define the mean settling velocity which is calculated as:

$$MSV = S \cdot D/100 \quad . \; . \; . \; . \; . \; . \; . \; (1)$$

where:

MSV = mean settling velocity
S = initial slope from Figure 3a
D = settling depth in settling chamber

The mean settling velocity can be used to compare settling rates under different environmental conditions and for different species (9) and to model algal dynamics in natural waters (7).

In describing settling in a rock filter, however, the nonlinear portion of Figure 3a is more important than the linear portion since predictions of removals greater than 50 percent are required. By assuming that the algal mass is proportional to fluorescence, the curve in Figure 3a can be transformed into the curves in Figure 4a as:

$$R = 100 - RF \quad . \; . \; . \; . \; . \; . \; . \; (2)$$

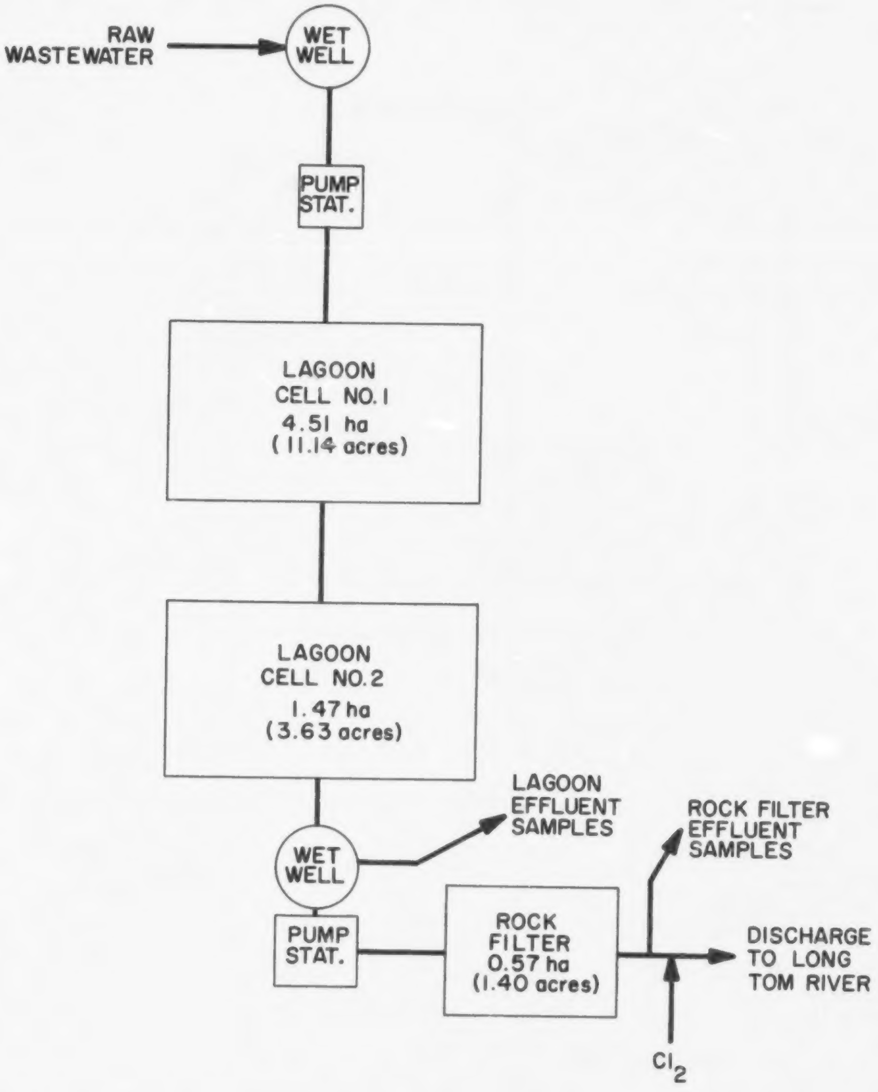

RAW WASTEWATER
WET WELL
PUMP STAT.
LAGOON CELL NO. 1 4.51 ha (11.14 acres)
LAGOON CELL NO. 2 1.47 ha (3.63 acres)
LAGOON EFFLUENT SAMPLES
WET WELL
PUMP STAT.
ROCK FILTER 0.57 ha (1.40 acres)
ROCK FILTER EFFLUENT SAMPLES
DISCHARGE TO LONG TOM RIVER
Cl_2

Figure 1. Schematic flow diagram of the wastewater treatment system at Veneta, Oregon.

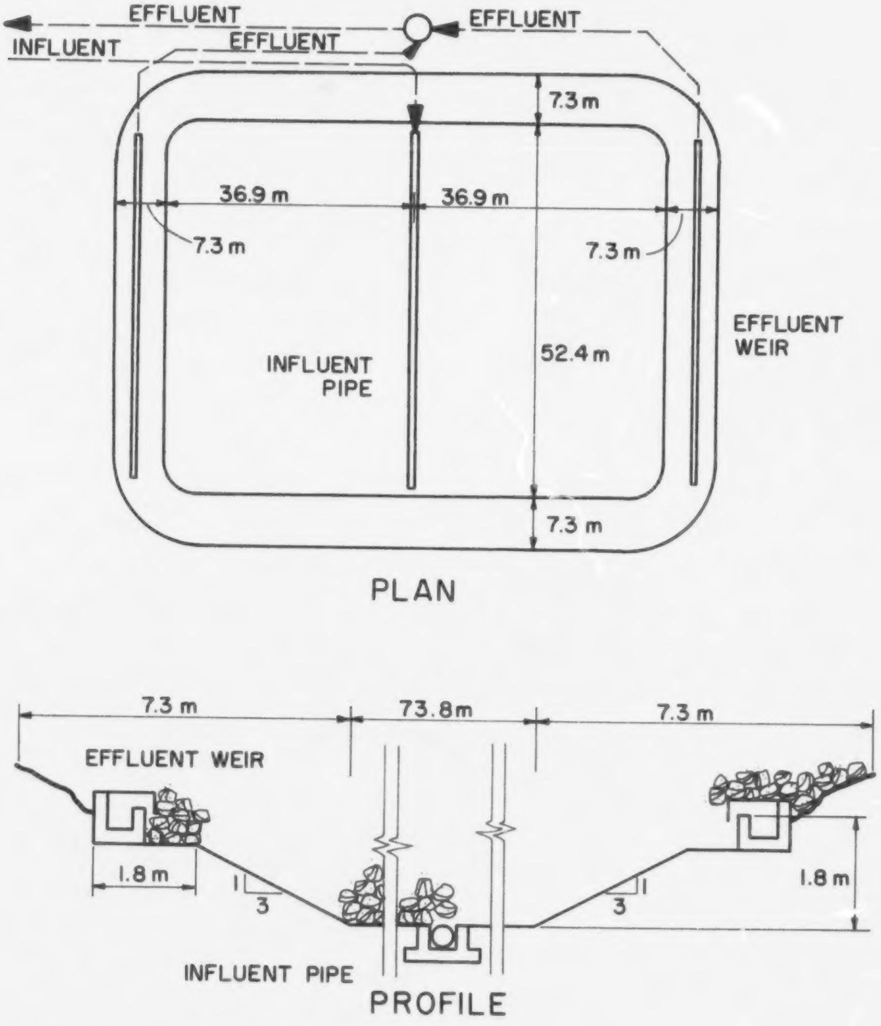
where:

R = percent removal
RF = relative fluorescence in settling test

The characteristic shape of Figure 3a gives a straight line portion in Figure 4a over a range of R from about 60 to 100 percent. This is the portion of the curve which is applicable to the design of rock filters. The transformation of the settling test data in Figure 3b is shown in Figure 4b; a linear response for R down to 45 percent was observed.

Figure 4 cannot be directly applied to the design of a rock filter because the effective settling depth in the rock filter is not equal to the settling depth in the settling chamber. The effective settling depth in the rock filter depends directly on the pore sizes which will be determined by the size and gradation of rocks. Studies are presently being conducted to estimate a settling depth for the Veneta rock filter.

It can be concluded from Figure 4 that algal sedimentation in the rock filter should be a linear function of 1/0. The proportionality coefficient is unknown. This relationship should hold for TSS, chlorophyll, and particulate BOD removal.


EFFLUENT
INFLUENT
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36.9 m
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INFLUENT PIPE
52.4 m
7.3 m
PLAN
7.3 m
73.8m
7.3 m
EFFLUENT WEIR
1.8 m
1
3
3
1
1.8 m
INFLUENT PIPE
PROFILE

Figure 2. Rock filter located at Veneta, Oregon.

SAMPLING AND ANALYTICAL METHODS

Samples were collected and analyzed for 7 consecutive days out of each month. This schedule was chosen to provide data on both long-term and day-to-day performance without over-extending available resources. Samples were collected from the rock filter influent (lagoon effluent) and the rock filter effluent as shown in Figure 1. Rock filter effluent samples were collected prior to chlorination so that interferences from chlorine would be eliminated in subsequent analysis.

A portable automatic sampler (ISCO model 1580) was installed at each sample point to collect 24-hour composite samples. The sample bottle in each sampler was packed in ice to bring sample temperature to 4°C or below. Composite samples were transferred to 1-liter plastic bottles and transported to the Corvallis laboratory for analysis of major wastewater constituents. Additionally, grab samples were collected for analysis of constituents whose concentration could be affected by automatic sampling or by storage for 24 hours. Grab samples were generally taken in the mid-afternoon. Temperature, dissolved oxygen, pH and sulfide analyses were performed on-site using grab samples.

Analysis of other constituents, including ammonia nitrate, suspended solids (total and volatile), organic nitrogen, phosphorus, chlorophyll, BOD_5, COD, and TOC were performed on 24-hour composite samples. Composite samples were preserved by refrigeration at 0°C and the addition of 0.8 ml H_2SO_4 per liter.

A summary of parameters analyzed, sample storage and analysis methods, and equipment used is given in Table 1.

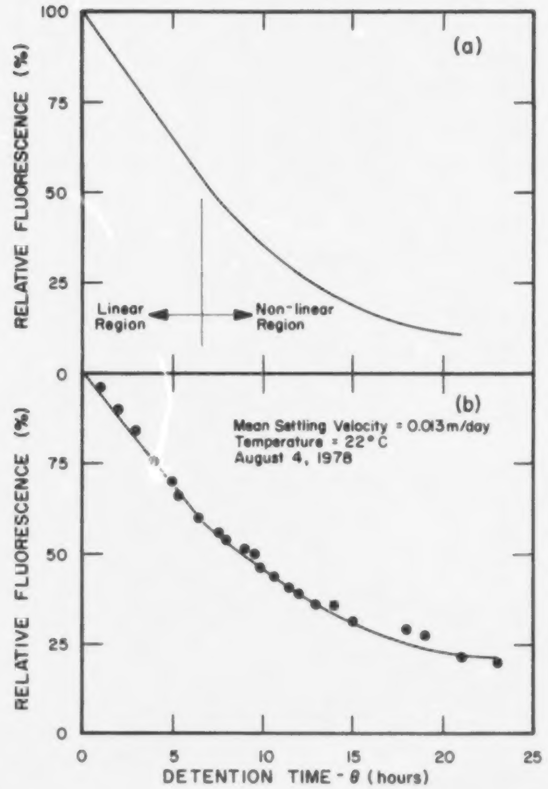

Figure 3. Algal removal versus detention time in settling test.

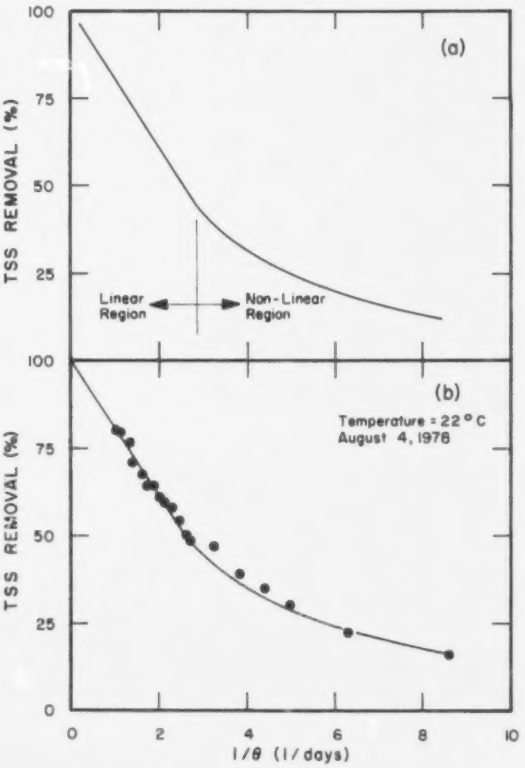

Figure 4. Algal removal versus inverse of detention time.

Table 1. Summary of analytical techniques.

Parameter	Type of Sample	Sample Storage and/or Preservation	Method of Analysis	Equipment Used	Reference
Temperature	Grab	On-site	Thermistor	YSI thermistor & YSI model 54 electronic meter	
DO	Grab	On-site	Membrane electrode	YSI DO probe & YSI model 54 electronic meter	Standard Methods, p. 450 (11)
pH	Grab	On-site	Electronic pH meter	Coleman Model 37A pH meter. Coleman glass & calomel electrodes	Standard Methods, p. 460
H_2S	Grab	On-site	Ion-specific electrode	Orion model 94-16A sulfide probe & model 407 specific ion meter	Orion Research (12)
NH_4^+	24-hour composite	On-site	Ion specific electrode Known addition method	Orion model 95-10 NH_3 probe & Orion model 407 meter	Standard Methods, p. 381
NO_3^- > 1 mg/1	24-hour composite	On-site	Ion specific electrode Known additional method	Orion model 92-07 NO_3^- probe model 407 specific ion meter	Orion Research (14) Yu & Berthouex (13)
< 1 mg/1	24-hour composite	Refrigeration @ 0°C + 0.8 ml H_2SO_4/1	Brucine method	Spectronic 20 spectrophotometer	
TSS & VSS	24-hour composite	On-site filtration	Gravimetric	GF/C Filter paper & Mettler Type H15 balance	Standard Methods, p. 94, 96
Organic N	24-hour composite	Refrigeration at 0°C	Kjeldahl digestion Acidimetric finish	Std. Kjeldahl apparatus	Standard Methods, p. 437
Phosphorus (total and soluble)	24-hour composite	On-site filtration Refrigeration @ 0°C + 0.8 ml H_2SO_4/1	Digestion with H_2SO_4 & HNO_3. Vanadomolybdophosphoric acid method	Micro-Kjeldahl digestion apparatus. Beckman model DB spectrophotometer	Standard Methods, p. 466
Chlorophyll	24-hour composite	Immediate filtration	Spectrophotometric	Heath Model EU-700 spectrophotometer	Standard Methods, p. 1029
BOD_5	24-hour composite	Immediate preparation	BOD bottle	300 ml BOD bottles YSI model - DO probe	Standard Methods, p. 543
COD	24-hour composite	Refrigeration at 0°C + 0.8 ml H_2SO_4/1	Dichromate digestion	Std. COD digestion apparatus	Standard Methods, p. 550
TOC	24-hour composite	Refrigeration at 0°C + 0.8 ml H_2SO_4/1	TOC analyzer	Oceanography Intl. Model 0524B TOC analyzer	Oceanography Intl. (15) Standard Methods, p. 532

RESULTS

Operational History

The City of Veneta is required to meet secondary treatment standards from November 1 to May 31 and is not allowed to discharge during the summer dry weather period (June 1 through October 31). Each spring the lagoon is drawn down to 0.76 m (2.5 ft) to allow for storage of all flows during the dry weather period. The rock filter is drained after discontinuance of discharge and left dry until the fall.

Discharge during the fall 1977 was begun on 11/10/77. The rock filter was filled with lagoon effluent and drained twice prior to startup. The sampling schedule used is shown in Figure 5.

Flooding of the adjacent Long Tom River during December 1977 restricted the discharge and resulted in ponding of the rock filter to about 0.3 m (1 ft) above the rock surface elevation. As a result, discharge and sampling were discontinued. Discharge was renewed in late December and sampling was begun on 1/1/78. The discharge rate was set at the maximum pumping capacity (∿ 400 gpm) due to the extremely high lagoon levels (> 2 m) and maintained at that rate through March.

Beginning in the February sampling period, a sulfide odor was apparent in the vicinity of the effluent manhole. However, sulfides could not be measured above the minimum detectable concentration of the sulfide probe (0.1 mg/l). The sulfide odor was reduced, but still noticeable in March, April, and May.

By mid-March, the high effluent pumping rate finally began to reduce the lagoon level. By 4/10/78, the lagoon was drawn down to the minimum water level of 0.76 m (2.5 ft) and discharge was discontinued. Discharge was renewed on 4/13/78 at ∿ 100 gpm and increased to ∿ 200 gpm on 4/17/78 during the April sampling period.

In the May sampling period, the lagoon level was at 0.9 m (3.0 ft) and an algal bloom was in progress.

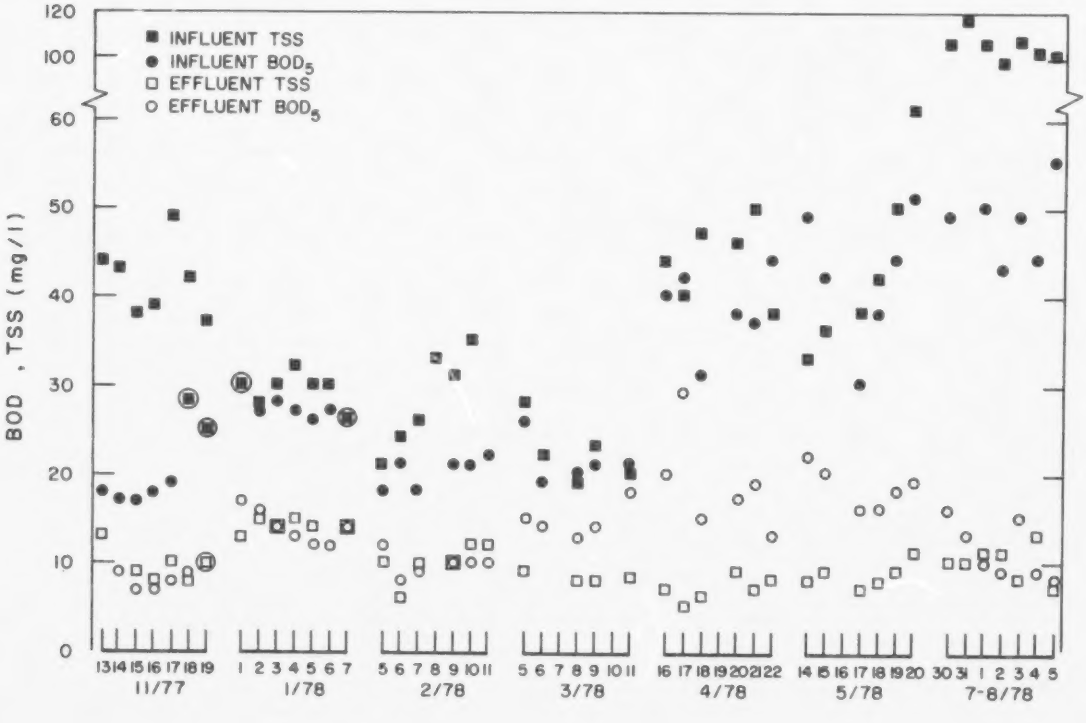

Figure 5. Influent and effluent BOD_5 and TSS over study period.

Lagoon effluent suspended solids were 33 mg/l at the beginning of the week and rose to 61 mg/l by the end of the week.

A variance of the city's discharge permit was requested of the Oregon Department of Environmental Quality (DEQ) to allow discharge during the summer dry weather period. This request was made so that data on summer operation of the rock filter could be obtained. This request was granted by the DEQ based on the following, more stringent, effluent limitations:

Parameter	Monthly Average	Weekly Average	Daily Average
A. BOD	10 mg/l	15 mg/l	20 mg/l
B. TSS	10 mg/l	15 mg/l	20 mg/l
C. Fecal Coliform	200/100 ml	400/100 ml	

D. pH within the range of 6.0 to 9 0

E. Effluent BOD concentration in mg/l divided by the dilution factor (ratio of receiving stream flow to effluent flow) shall not exceed 1.

Due to the low lagoon level and the high evaporative rate during the summer, however, discharge was not begun until 7/21/78 when the lagoon level reached 1.2 m (4.0 ft). By this time, a bloom of blue-green algae had caused the lagoon effluent TSS to exceed 100 mg/l.

Data and Data Analysis

Daily rock filter influent and effluent BOD_5 and TSS values over the seven sampling periods completed to date are shown in Figure 5. Good suspended solids removals (> 70 percent) were obtained beginning with the first sampling on 11/13/77, 3 days after the fall startup. This supports the theory that the removal mechanism is primarily physical (settling) and not biological.

Percentage removals of suspended solids were lower during the winter months of January, February, and March due to the high effluent pumping rate,

Table 2. Data summary for rock filter.

Parameter	Weekly Averages													
	11/13/77 to 11/20/77		1/1/78 to 1/7/78		2/5/78 to 2/11/78		3/5/78 to 3/11/78		4/16/78 to 4/22/78		5/14/78 to 5/20/78		7/30/78 to 8/5/78	
	I[a]	E[a]	I	E	I	E	I	E	I	E	I	E	I	E
Hydraulic Loading (1/days)	0.13		0.28		0.28		0.27		0.12		0.17		0.07	
Temperature (°C)	8.4	8.6	6.0	6.3	9.3	9.0	10.6	9.8	12.3	11.5	18.2	17.0	22.0	20.8
pH	9.0	7.7	8.6	7.1	7.6	7.1	7.9	7.1	8.7	7.2	9.9	7.3	9.6	7.6
DO (mg/l)	10.1	4.5	15.4	6.2	11.2	3.2	10.8	3.0	10.8	3.2	17.4	2.9	6.9	1.8
TSS (mg/l)	42	9	29	14	28	10	22	9	44	7	43	9	105	10
TVSS (% of TSS)	83	89	90	89	90	86	89	86	92	66	84	85	88	67
BOD_5 (mg/l)	20	9	27	14	20	10	21	15	39	19	42	18	43	11
COD (mg/l)	121	77	67	45	51	36	61	44	147	80	159	104		
TOC (mg/l)	37	28	24	16	19	13	23	16	-	-	56	39		
NH_4^+-N (mg/l)	0.8	1.7	3.5	2.9	15.5	12.4	15.9	14.3	3.8	5.5	2.6	7.2	0.2	3.5
Org-N (mg/l)	4.1	3.4	5.8	5.4	5.7	1.4	3.9	3.3	8.8	4.5	8.4	5.2		
NO_3^--N (mg/l)	1.5	2.1	1.0	1.6	0.8	1.1	1.1	0.8	1.7	1.5	2.3	1.0	1.9	1.2
Soluble P (mg/l)	4.8	3.9	1.7	1.6	2.5	3.1	2.1	3.5	6.0	4.6	3.7	4.9		
Total P (mg/l)	5.2	4.1	2.1	1.6	3.2	3.4	2.7	3.1	6.8	5.0	5.6	5.4		
Chlorophyll a (g/l)	-	-	340	160	260	72	160	32	690	13	210	34		
Chlorophyll b (g/l)	-	-	39	23	59	15	17	3	348	12	240	38		
Chlorophyll c (g/l)	-	-	23	15	29	6	19	5	148	22	380	80		
Sulfides (mg/l)	ND	ND	ND	ND	ND	<0.1	ND	<0.1	ND	ND	ND	ND		

[a] I = rock filter influent, E = rock filter effluent
ND = not detectable (<< 0.1 mg/l)

but the lower algal concentrations in the lagoon effluent during these months resulted in final effluent BOD_5 and TSS values of less than 20 mg/l on all days sampled.

The rock filter effluent TSS did not exceed 15 mg/l and the BOD_5 did not exceed 20 mg/l over the study period, except on 4/17/78, when a BOD_5 of 29 mg/l was recorded. This value was much higher than all other BOD_5 values reported that week, however, and may have been an analytical error. The low lagoon levels during the spring resulted in increased soluble BOD_5 values that were not removed in the rock filter. As a result, higher effluent BOD_5 values were noted.

A summary of all rock filter influent and effluent data completed to date is shown in Table 2. These data are averages of the 7 consecutive days of sampling during each sampling period. The hydraulic loading indicated was the pumped flow rate divided by the total rock filter volume (m^3 water/m^3 rock filter-day or 1/days) Water temperature increased slightly in passing through the rock filter during the winter months, but decreased during the summer. The pH of the lagoon effluent approached 10 during summer periods of heavy photosynthesis, but the rock filter provided sufficient detention time without photosynthetic activity so that dissolved carbon dioxide was replenished and the pH lowered to between 7 and 8.

As mentioned previously, dissolved oxygen (DO) readings were taken on grab samples, generally in the mid-afternoon. Therefore, the influent (lagoon effluent) DO values do not represent the diurnal variations in DO which occur in lagoons. The rock filter undoubtedly dampens these diurnal variations to a large extent; therefore, the effluent DO values shown are probably representative of average conditions.

Ammonia-nitrogen was observed to increase noticeably in passing through the rock filter during the warmer months (Figure 6). This is apparently the result of increased biological degradation rates at these times. Due to the existence of both aerobic and anoxic

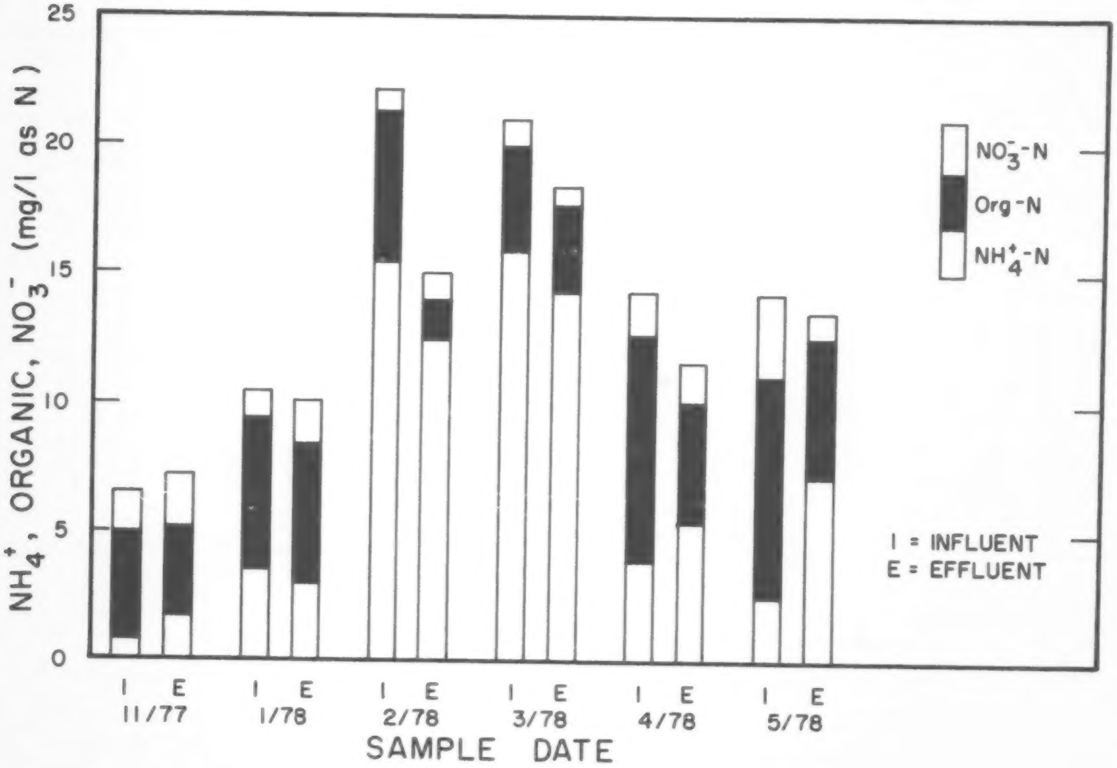

Figure 6. Weekly averages of NH_4^+-N, Org-N and NO_3^- for rock filter.

zones in the rock filter at most times, both nitrification and denitrification are possible, even simultaneously. A small amount of nitrification appeared to occur during the colder months, changing to an apparent predominance of denitrification in the summer. The amount of biological nitrification and denitrification which can be supported in a rock filter, however, is definitely limited; nitrification by the limited oxygen content in the influent stream and denitrification by variations in the extent of the anoxic zone.

Chlorophyll concentrations are indicative of viable algae and removal can be directly correlated with suspended solids removal as shown in Figure 7. Variations in the proportion of chlorophylls a, b, and c between sampling periods are indicative of changes in dominant algal species. Additionally, beginning in March 1978, grab samples were analyzed once each sampling period for algal species and counts. Results of this analysis are contained in Appendix A.

Correlation with Settling Theory

For discretely settling particles, the percentage removal of suspended solids should be a direct function of hydraulic loading rate. This appears to be the case for the removal of suspended solids in the rock filter as shown in Figure 8. This relationship is supported by the measured settling curves for algae (see Figures 3 and 4). Effluent suspended solids and BOD$_5$ showed no correlation with hydraulic loading rate (Figure 9) as would be expected based on the relationship shown in Figure 8.

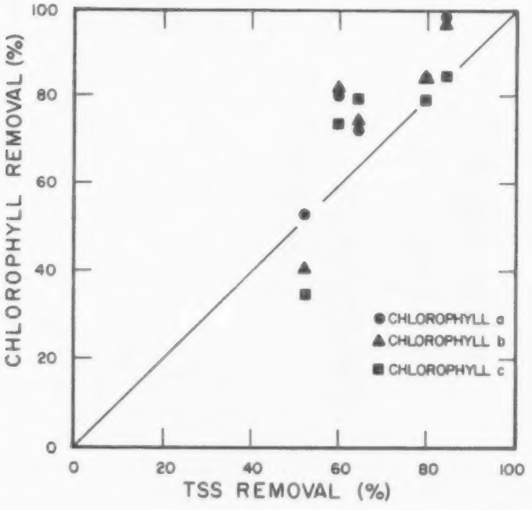

Figure 7. Correlation of chlorophyll removal to TSS removal.

In Figure 8, the three points of highest hydraulic loading correspond to January, February, and March 1978 when the lagoon level was extremely high and effluent pumping was maintained at the maximum rate (400 gpm). The point of lowest hydraulic loading corresponds to August 1978 when a bloom of blue-green algae had occurred in the lagoon. These data points represent widely varied temperatures (6-22°C) and algal species composition (see Appendix A). Since those points deviate only slightly from the straight line relationship in Figure 8, it was concluded that the settling rates of algal populations present in the Veneta lagoon do not vary substantially with changes in dominant species, temperature, or other environmental conditions. Apparently the decreased settling rate due to lower tempratures is compensated by an equal increase in the mean settling rate of the algal species present under colder environmental conditions. If the lack of temperature and species composition effects is universally true, then this could greatly simplify the design of rock filters.

In accordance with settling theory, BOD removal in the rock filter should be through the removal of particulate BOD, primarily algae. This relationship is shown in Figure 10 as a correlation of BOD$_5$ and TSS removal. The straight line suggests a removal of about 0.5 mg BOD$_5$ per mg of suspended solids.

CONCLUSIONS

Based on the results of this study, it is concluded that:

1. The Veneta rock filter can consistently meet a daily maximum effluent limitation of 20 mg/l TSS and 20 mg/l BOD for hydraulic loadings of less than 0.30 m^3/m^3-day.

2. A well-designed and operated rock filter-lagoon system could apparently meet effluent limitations of 10 mg/l maximum BOD and TSS based on monthly averages.

3. The primary removal mechanism in the rock filter is sedimentation which results in a linear correlation between TSS removal efficiency and hydraulic loading rate over a removal efficiency range of about 50 to 100 percent.

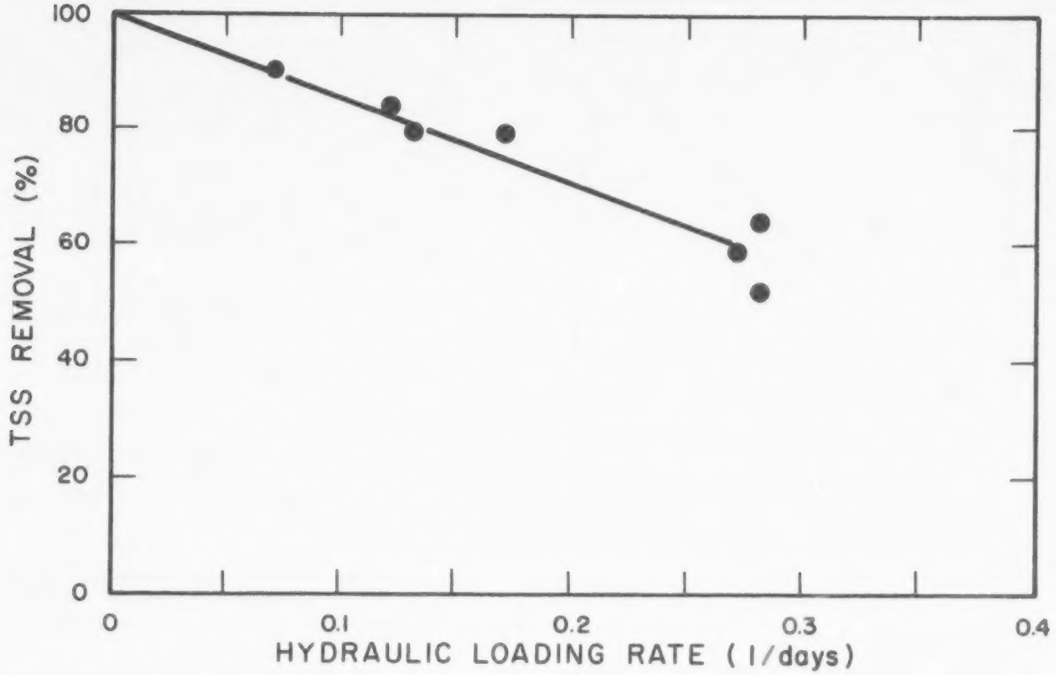

Figure 8. Total suspended solids removal versus hydraulic loading rate.

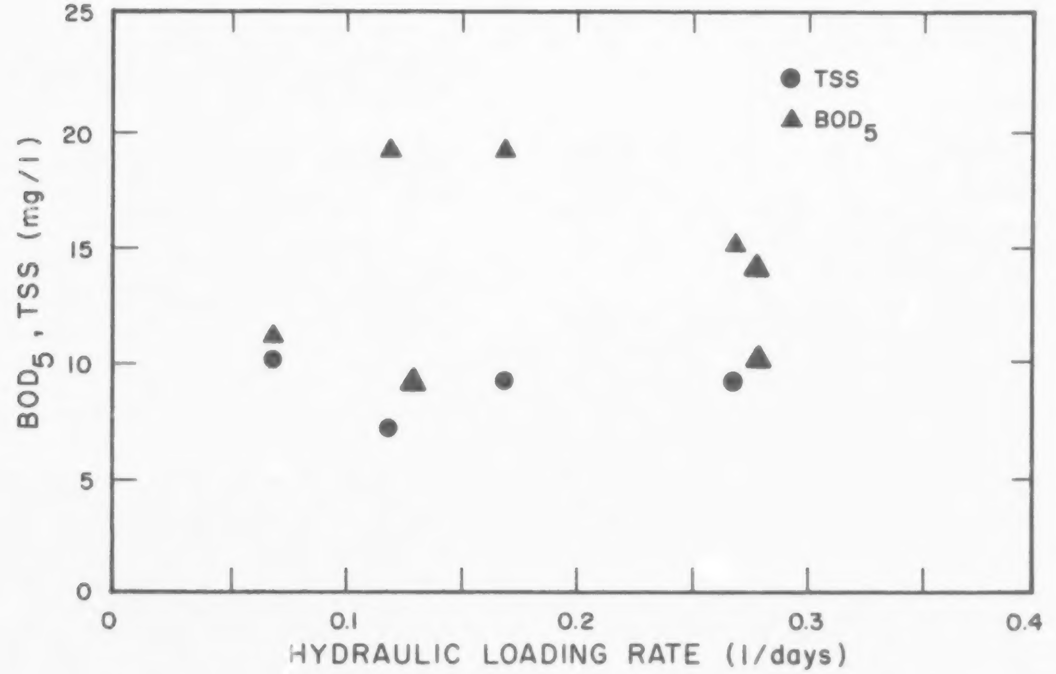

Figure 9. Effluent quality from rock filter as a function of hydraulic flow rate.

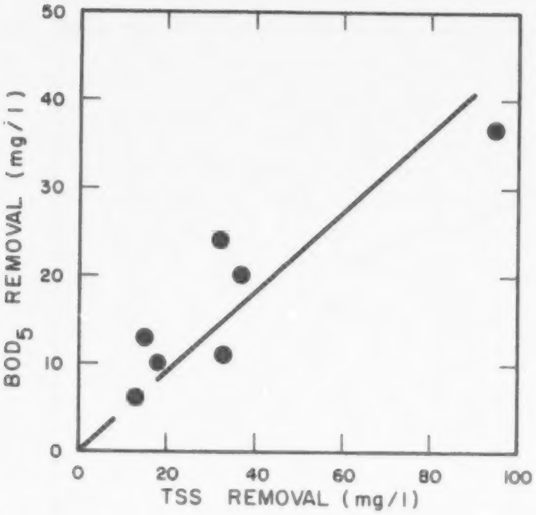

Figure 10. Correlation of BOD_5 removal to TSS removal.

4. BOD removal in the rock filter primarily results from settling of particulate BOD with little net removal or generation of soluble BOD.

5. Rock filters appear to be compatible with the capabilities of typical lagoon operators.

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APPENDIX A

Table 3. Algal speciation for grab samples--March 9, 1978.

	Rock Filter Influent		Rock Filter Effluent	
Taxon	genus-species	Rel. Abundance	genus-species	Rel. Abundance
Chlorophyta	Scenedesmus acuminatus	0.195	Scenedesmus acuminatus	0.139
	Chlorella vulgaris	0.189	Chlorella vulgaris	0.458
	Pyraminonas cf. tetrarhynchus	0.166	Ankistrodesmus Falcatus	0.125
	Ankistrodesmus Falcatus	0.122	Chlamydomonas cf. globosa	0.028
	Chlamydomonas cf. globosa	0.047	Actinastrum hantzschii	0.014
	Coccomonas sp.	0.030		
	Scenedesmus quadricauda	0.006		
	Actinastrum hantzschii	0.006		
Cyanophyta	Schizothrix calcicola	0.018		
Euglenophyta	Astrasia sp.	0.106	Euglena sp. 1	0.042
	Euglena sp. 1	0.059	Trachelomonas volvocina	0.180
	Euglena sp. 2	0.024	Trachelomonas sp. 1	0.014
	Trachelomonas volvocina	0.012		
Pyrrhophyta	Glenodinium sp.	0.030		

Table 4. Algal speciation for grab samples--April 20, 1978.

	Rock Filter Influent		Rock Filter Effluent	
Taxon	genus-species	Rel. Abundance	genus-species	Rel. Abundance
Chlorophyta	Chlamydomonas cf. globosa	0.99	Chlamydomonas cf. globosa	0.44
	Scenedesmus acuminatus	0.01	Scenedesmus acuminatus	0.11
			Chlorella vulgaris	0.11
			Actinastrum hantzschii	0.06
Euglenophyta			Euglena sp. 1	0.17
			Trachelomonas sp. 1	0.11

Table 5. Algal speciation for grab samples--May 18, 1978.

	Rock Filter Influent		Rock Filter Effluent	
Taxon	genus-species	Rel. Abundance	genus-species	Rel. Abundance
Chlorophyta	Tetraedron regulare	0.75	Tetraedron regulare	0.83
	Chlorella vulgaris	0.05	Scenedesmus acuminatus	0.07
	Scenedesmus acuminatus	0.12	Ankistrodesmus Falcatis	0.07
	Scenedesmus quadricauda	0.04		
	Ankistrodesmus Falcatus	0.01		
Chrysophyta	Nitzschia sp.	0.02		
Pyrrhophyta			Gilenodinium sp.	0.02

Table 6. Algal speciation for grab samples--August 2, 1978.

	Rock Filter Influent		Rock Filter Effluent	
Taxon	genus-species	Rel. Abundance	genus-species	Rel. Abundance
Cyanophyta	Anacystis cyanea (Microcystis Flos-aquae)	1.00	Anacystis cyanea	0.999
			Schizothrix calcicola	0.001

COMMENT ON FIELD EVALUATION OF ROCK FILTERS FOR REMOVAL OF ALGAE FROM LAGOON EFFLUENTS

M. L. Cleave*

Williamson and Swanson (1978) assume that discrete settling is responsible for removal of algal particles from lagoon effluents. The authors use chlorophyll extractions and relative fluorescence (an estimate of chlorophyll) as a measure of the efficiency of treatment by the rock filter. Although discrete settling of algae is assumed for lakes and ponds (Bella, 1970), the validity of this assumption for rock filter settling of the Veneta lagoon effluent is questionable because of the type of algae and the biomass measurement technique.

The type of algae contained in the Veneta lagoon effluent was identified via a relative abundance index on algal speciation conducted on grab samples from the lagoon effluent. The algal speciation listing of the lagoon effluent on March 9, 1978, states that 47 percent of the algae present were motile. Motility would render these algae unsusceptible to sinking. Also, 10 percent of the algae, represented by the genus *Astasia*, contained no chlorophyll. The absence of chlorophyll would render this alga unmeasurable by either fluorescence or chlorophyll biomass indicators. The algal speciation listing of the lagoon effluent for April 20, 1978, states that 99 percent of the algae present were motile. This assumes that the *Chlamydomonas sp.* listed was not in a *palmelloid form* since this was not noted as such. The algal speciation listing of the lagoon effluent for August 2, 1978, states that the alga of the lagoon effluent was 100 percent *Microcystis flos-aquae* (*Microcystis aeruginosa Kuetz; emend.* Elenkin 1924). This blue-green alga contains gas vacuoles which allow it to float to the surface of water independent of turbulence (Fogg et al., 1973).

It was shown by Titman and Kilham (1976) that in general, when comparing different algal species in the same physiological condition, the larger algal species display a tendency to sink more rapidly than the smaller algal species. Many previous investigators have shown that algal sinking rates are not a species specific constant because nutrient depleted cells sink 2 to 4 times more rapidly than nutrient enriched cells (Smayda, 1970, 1974; Smayda and Boleyn, 1965, 1966; Boleyn, 1972; Eppley et al., 1967). Therefore, settling rates of algae should be sensitive to either aerobic or anaerobic dark incubation among other nutrient/chemical variations.

The design of the rock filter is based on algal settling tests where Newton's law would be expected to apply. Assuming that either Newton's or Stokes' law applies, the settling rates of algae are temperature dependent by definition. Also, the algal suspension is placed in a fluorometer and exposed to intense light for prolonged periods of time during the settling tests. Thus, the settling curves could be artifacts of either chloroplast reorientation for the eucaryotic algae or gas vacuole degradation in the blue-green (procaryotic) algae. Kiefer (1973) discussed the two components of fluorescent decay due to movement and redistribution of chloroplasts within eucaryotic algal cells as shown in Figure 1. This curve displays the same general conformation as the settling curve presented for algal removal by Williamson and Swanson (their Figure 3). Dinsdale and Walsby (1972) showed that when the blue-green alga, *Anabaena flos-aquae*, was transferred from low to high light intensity, in the presence of carbon dioxide-containing air, the turgor pressure increased by 100 kilonewtons per square meter or more. This was sufficient to cause the collapse of gas vacuoles after which the cells lost their buoyancy. Because there is no light present in the rock filter at the time of algal settling, this fluorometric technique may not be applicable to the determination of mean settling velocities for algae in rock filters.

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*M. L. Cleave is a graduate student, Utah State University, Logan, Utah.

Williamson and Swanson concluded that settling rates of algal populations from Veneta lagoon were independent of dominant species, temperature, or other environmental conditions. This appears to be an over simplification of algal settling mechanisms. Rather it would suggest that the removal mechanism of algae in rock filters is not discrete settling. Therefore, discrete algal settling would not appear to be a valid basis for the design of rock filters. Other settling models should be investigated; however, other biological processes may in fact control removal.

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AUTHOR'S RESPONSE TO COMMENTS BY M. L. CLEAVE

Kenneth J. Williamson and
Gregory R. Swanson

The authors wish to thank Ms. Cleave for her comments regarding "Field Evaluation of Rock Filters for Removal of Algae from Lagoon Effluents." We agree that settling may not be the only algal entrapment mechanism in rock filters, but maintain that settling is the primary mechanism. Clearly, many biological and physical removal mechanisms are occurring in this treatment system.

Although limited data exist on the variation of algal species in lagoons by season and geographical location, it appears that the predominant algae are nonmotile green algae. Allen (1955), for example, surveyed several California lagoons over a two year period and found Chlorella and Scenedesmus to be usually dominant. These algae depend upon wind-induced circulation to maintain themselves in the photic zone and will settle under quiescent conditions, (Fogg, 1975). Stutz-McDonald and Williamson (1978) measured mean settling velocities on pure cultures of Scenedesmus acuminatus Lager and Chlorella vulgaris Beij, as 0.26 and 0.14 m/day, respectively, at 21°C. Settling rates were dependent on temperature as expected; however, neither aerobic dark nor anaerobic dark incubation significantly affected settling rates.

Blue-green algae, such as Microcystis, occur to a minor extent in lagoons during most of the year but, under the right environmental conditions, may undergo massive blooms in hot, summer months. This was the case at the Veneta lagoon this past summer. These algae generally contain gas vacuoles for bucyancy regulation. Although the mechanism regulating the formation of gas vacuoles is not entirely clear, gas vacuoles develop most abundantly (causing flotation) in dim light and collapse as a result of photosynthesis, often resulting in a diurnal up-and-down movement. Also, higher concentrations of nutrients may favor gas vacuole formation (Fogg et al., 1973). Under the dark, nutrient-rich conditions which exist in the rock filter, therefore, evidence indicates that gas vacuole formation and subsequent flotation could occur.

Physical observation of recent fluorometric settling tests on Veneta lagoon samples with Microcystis flos-aquae still predominating show that most of the algae depart from the optical window by flotation to the surface rather than settling. It therefore appears that the entrapment mechanism for these algae is probably natural biological flotation rather than settling. This phenomenon can be viewed essentially as inverse settling in that these algae are trapped below the rocks in their attempt to rise, where they eventually die and decay. Also, in support of this hypothesis, a recent physical investigation of the rocks at and below the water surface in the rock filter showed that algal residues can now be found on the bottom surfaces of the rocks and that a brown scum layer (1-4 mm thick) of algal residue has formed at the water surface.

Phytoflagellates (motile algae) such as Euglena and Chlamydomonas are often dominant at the lagoon edges, where little mixing occurs, but generally comprise only a small part of the total algal population (Allen, 1955). During the two sampling periods at the Veneta lagoon when phytoflagellates were dominant (March and April, 1978) the relative abundance indices indicated that these algae were removed as well as, or better than other algal species in passing through the rock filter. The removal mechanism for phytoflagellates, however, is not clear. Dinoflagellates studied by Eppley et al. (1968) were found to swim at rates of 1 to 2 meters per hour, a more than sufficient rate to escape settling. However, these species may lose their motility and settle under extended dark conditions present in the rock filter.

In terms of the total removal mechanism, the rock filter can be likened to a clarifier used in large-scale wastewater treatment. In a clarifier, the detention time provided allows settleable solids to settle and floatable solids to rise to the surface. In the rock filter, nonmotile green algae will settle and blue-green algae with gas vacuoles will rise. The major difference is that the rocks in the rock filter reduce the settling or rising distance to a few centimeters. Because the settleable algae generally are dominant and because the very low settling velocities associated with these algae probably control rock filter performance in most instances, a settling model appears to be appropriate for design of rock filters. Such a model will not predict rock filter performance when blue-green algae are dominant in the lagoon, no more than settling models for clarifiers predict removal rates under conditions of bulking or rising sludge.

The authors must disagree with Ms. Cleave's contention that the fluorometric technique is not appropriate for measuring algal settling velocities. While approximate and not all-inclusive of algal species, the settling velocities obtained appear to be reasonable and correlate well with removal efficiencies obtained in both the full-scale and the pilot-scale rock filters. The data presented by Ms. Cleave on reduction in fluorescence due to movement and redistribution of chloroplasts offers no explanation of settling curves obtained with the fluorometer. On several occasions, the samples have been removed from the fluorometer following settling tests, remixed, and then replaced. The fluorescence reading obtained by this test has always been nearly equal to the initial fluorescence reading.

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MICROSCREENING AND OTHER PHYSICAL-CHEMICAL TECHNIQUES FOR ALGAE REMOVAL

Richard A. Kormanik and Joe Bob Cravens*

INTRODUCTION

Much research has been conducted utilizing dissolved air flotation or various types of sand filtration techniques to remove algae from lagoons during the warm months of the years. Several full scale installations have been built using dissolved air flotation (DAF) with chemical coagulation for the removal of algae from ponds. The most recent installation using DAF is Sunnyvale, California. Operating experience at this time is not available. DAF units have been in operation for several years, removing algae at a lagooning operation in Stockton, California; however, exorbitant operating costs are plaguing that operation.

The writers of this text have conducted DAF studies as well as microscreening studies for the removal of algae from lagoons. Historically, DAF has been proven to be an effective means of removing algae from lagoon effluent with the use of alum as a coagulant. However, this method has shown to have expensive operating costs because of the need for chemical feed systems as well as a method for disposing of the algae/alum sludge.

An alternative to DAF which was thoroughly investigated in the field, is microscreening using 1 micron polyester fabric (1). Work done by Kormanik and Cravens (2) describes full scale field research done using microscreening with 1 micron polyester fabric. Previous investigations using microscreens were conducted using 23 and 25 micron stainless steel media. Golueke et al. (3) conducted pilot-plant studies; however, removal efficiencies were nominal. Drum speeds were 10, 20, and 30 rpm.[1] The algae species most predominant in lagoons are Scenedesmus, Chlorella, and Chlamydomonas (see Figure 13 for algae bioassays).

In 1975, Caldwell et al. (4) and Parker (5) tested microscreening on an oxidation pond effluent; however, Chlorella and Scenedesmus algae were the predominant species present. Typically, the size distribution range for these species is 1-5 micron; thus, results were marginal when using the 23 micron media.

Rupke and Chisholm (6) reported the use of a microscreen pilot unit with 23 micron stainless steel media to upgrade sewage lagoon effluents. During July through September SS reductions were from 20 to 50 percent and BOD reductions were 10 to 20 percent without coagulants. With coagulants such as alum, ferric chloride, or polyelectrolytes, suspended solids capture was as high as 70 percent SS.

Essentially algae are present in ponds in two general classifications:

Chlorophyta (Green)
Cyanophyta (Blue Green or mat formers)

Each of the groups found in lagoons contain free swimmers and nonmobile species and are in size range of 1 to 10 microns.

Algae growth in ponds vary in quantity and specie. During ideal pond conditions, green algae are predominant. These algae are more uniformly dispersed throughout the pond. Thus, the ideal symbiotic relationship is achieved when nutrients released via bacterial decomposition are assimilated by the algae. In turn the algae release oxygen via photosynthesis and support aerobic decomposition.

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[1] Microscreen submergence time is too short at speeds of 10-30 rpm.

Blue green algae form in stagnant pond areas, or during organic shock loading periods. These floating mats minimize sunlight penetration and retard the desired green algae growth. The floating mats entrap organics, dead algae, etc., and the characteristic "pig pen" odors result.

Diurnally, algae tend to concentrate near the surface during sunlight productive hours and die off and settle out during the night.

Again, Figure 13 illustrates typical green and blue green algae species in ponds. Note the typical size, ranging, in general, from 1-10 microns. Since numerous cell fragments and juvenile algae are present, effective removal via microscreening requires the 1 micron media size.

In 1976 Envirex initiated a field test program using a 4 foot diameter by 2 foot long trailer-mounted microscreen. The unit is fully automated, including on-line suspended solids monitoring and recording meters for influent and effluent. Hydraulically, the unit can handle 10-200 gpm, an appropriate range for 1, 6, 10, and 21 micron polyester media (Figures 1 and 2).

Figure 3 illustrates the polyester media, magnified 500 times, as compared to Figure 4, stainless steel media. The polyester media is suited for high headloss (12" or greater), is not subject to fatigue, and is inert to chlorination required for slime control. The polyester has a consistent weave for uniform aperture size with maximum open areas.

To date extensive tests have been conducted at seven pond sites. Table 4 is a summary of the test program, identifying microscreen influent and effluent SS and BOD concentrations.

At one of the field sites, DAF and microscreening were investigated and compared. Table 1 shows this comparison. Both processes obtain a 30/30 (BOD/SS) standard. The costs for these two process approaches are compared later in this text. Table 1 also shows typical experienced operating conditions for the DAF. The results of both types of processes show that a 30/30 effluent can be obtained.

One interesting observation was the high pH fluctuation over a 24-hour period. Figure 5 shows observed pH variation. What is significant is that the alum dosage must be varied to meet the variations in pH as well as the amount of suspended solids (algae). This dual process variation causes significant operating expenses as experienced by other installations.

MICROSCREENING

Kormanik and Cravens (1,2) have conducted 2 years of full scale field studies using microscreens for the removal of algae from lagoons. Figures 6, 7, 8, 9, 10, 11, and 12 summarize the additional field studies for the removal of algae (as recorded as suspended solids) as well as the BOD associated with the algae. These extensive studies using a full scale microscreen have shown that the removal of algae using 1 micron polyester fabric is a viable process alternative for the removal of algae.

At one of the sites researched, Peoples and Cravens (7) conducted tests and comparative data were obtained using microscreening and existing site pressure filters. Table 2 shows the comparative results. Here microscreening is shown to be a more effective means of removing SS and the associated BOD. Table 3 shows the SS/BOD relationships before and after microscreening. It can be noted that from all of the sites researched, it appears that significant algae removal (measured as SS) must be attained in order to obtain a BOD level of 30 mg/l. Note that only one lagoon tested (Site No. 6) had an acceptable BOD/SS (30/90). Table 4 shows tabular results of all of the lagoons tested for algae removal. This study has shown that microscreening with 1 micron polyester fabric is a process alternative to DAF and sand filtration. In addition, no operational problems were encountered at any of the sites tested. This process success, as well as the ease of operation, will show the cost effectiveness of the microscreen for algae removal.

COST EFFECTIVE ANALYSIS

As mentioned before, dissolved air flotation (DAF) with alum has been shown to be a successful process for the removal of algae. The 2 year study (1,2) using microscreening with 1 micron polyester fabric as presented here, shows that it, too, is a process alternative. What is necessary is to demonstrate which alternative is cost effective for the removal of algae.

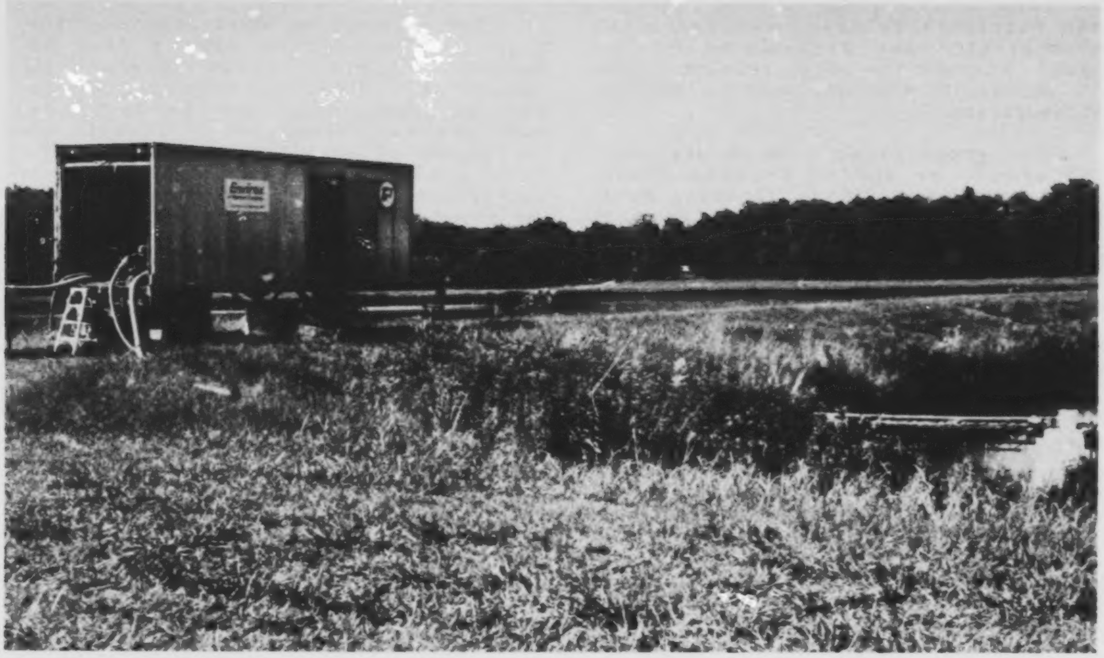

Figure 1. Microscreen test van at a typical lagoon site.

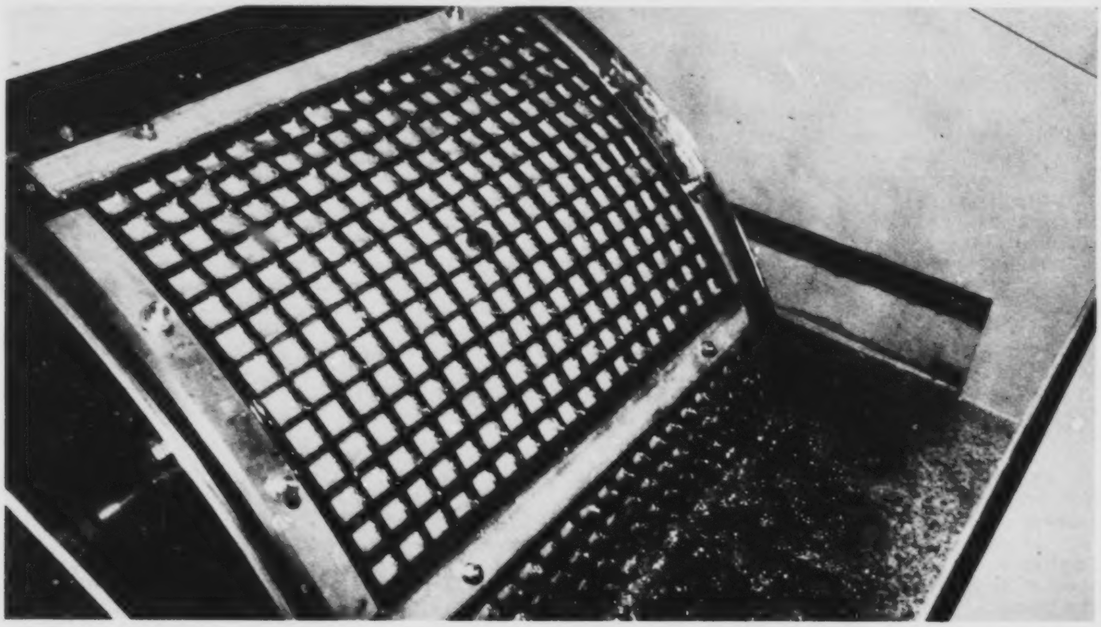

Figure 2. Envirex 4 foot diameter x 2 foot long microscreen.

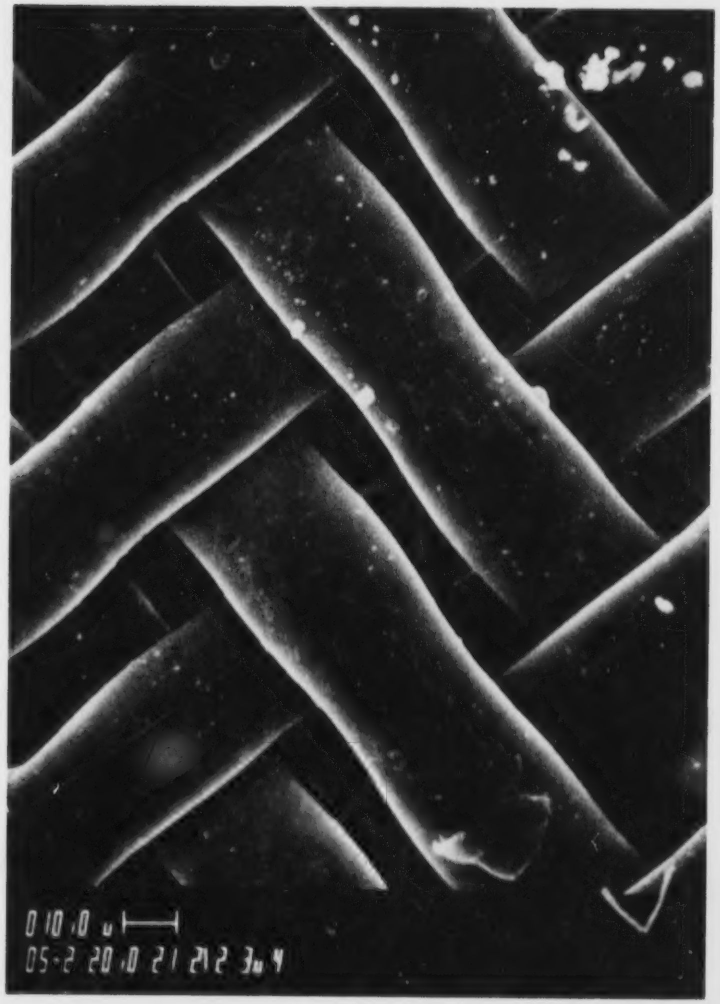
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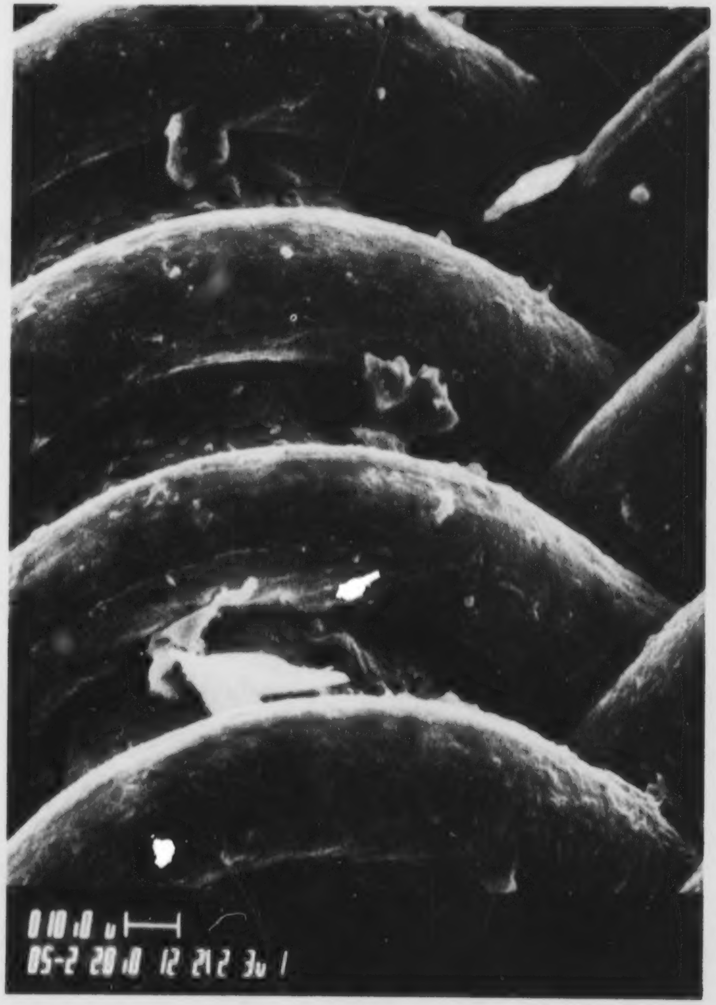
Figure 3. Electron-photomicrograph 21 micron polyester media (500X).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05-2 20.0 12 21 2 3u 1

Figure 4. Electron-photomicrograph 21 micron stainless steel media (500X).

Table 1. Summary of side by side microscreen vs. dissolved air flotation continuous 24 hour runs.

Dissolved Air Flotation Operating Conditions	Pond Effluent (mg/l)	Effluent (mg/l)	
		DAF	MICROSCREEN
	T-BOD/TSS	T-BOD/TSS	T-BOD/TSS
100 mg/l ALUM NO FLOCCULATION 25% RECYCLE OVERFLOW RATE 2 gpm/ft^2	29/37	9/12	23/4
100 mg/l ALUM NO FLOCCULATION 25% RECYCLE 3 gpm/ft^2	35/47	13/23	23/19
100 mg/l ALUM 11 MIN. FLOCCULATION 25% RECYCLE 3.4 gpm/ft^2	47/61	—/23	4/14
145 mg/l ALUM 10.5 MIN. FLOCCULATION 19% RECYCLE 3.2 gpm/ft^2	-	16/20	3/17
100 mg/l ALUM 19 MIN. FLOCCULATION 25% RECYCLE 2 gpm/ft^2	42/58	15/20	33/20
100 mg/l ALUM 3 mg/l POLYMER 7.5 MIN. FLOCCULATION 20% RECYCLE 3.6 gpm/ft^2	44/34 SOLUBLE BOD 18	9/4 SOLUBLE BOD 4	29/10 SOLUBLE BOD 4

Table 2. Summary of side by side microscreen vs. pressure filter continuous 24 hour test runs.

Run Number	Microscreen Influent Source	Hydraulic Loading (gpm/ft^2)		Suspended Solids (mg/l)		
		Microscreen	Pressure Filter	Microscreen & Pressure Filter Influent	Micro-screen Effluent	Pressure Filter Effluent
1	Polishing Pond Effluent	2.40	2.06	29	7	21
2	Polishing Pond	2.90	1.93	27	14	24
3	Polishing Pond	1.65	1.93	19	4	13
4	Polishing Pond	1.72	1.93	40	4	22
5	Polishing Pond	2.50	-	28	10	24

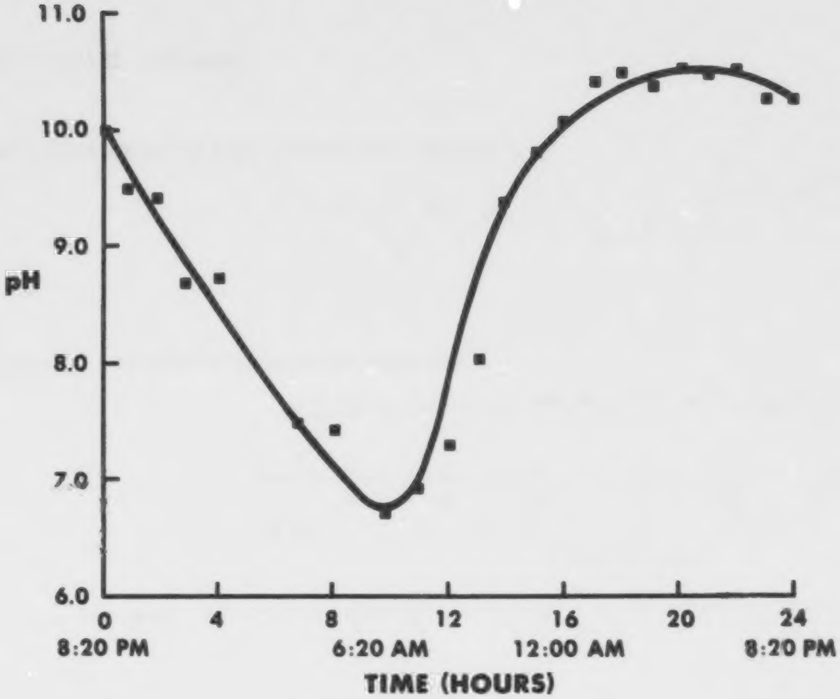

Figure 5. pH vs. time at Gardendale, South Carolina.

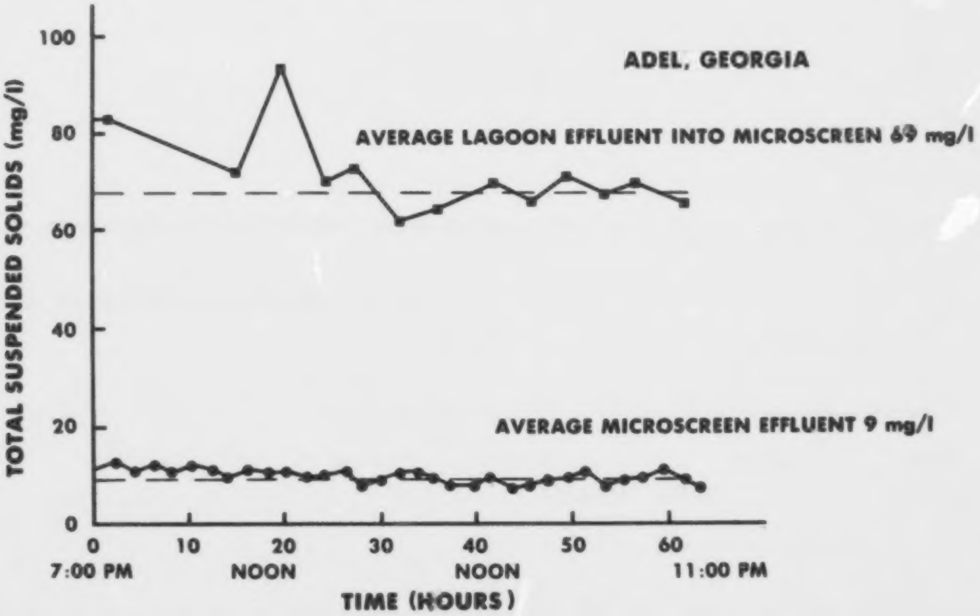

Figure 6. Microscreen influent and effluent suspended solids vs. time using 1 micron media.

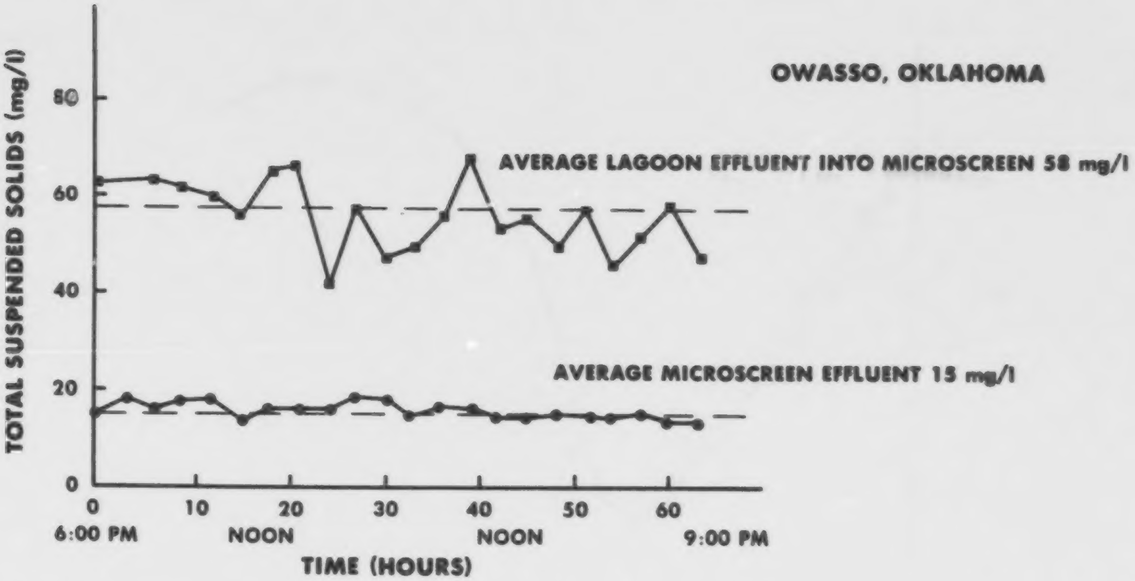

Figure 7. Microscreen influent and effluent suspended solids vs. time using 1 micron media.

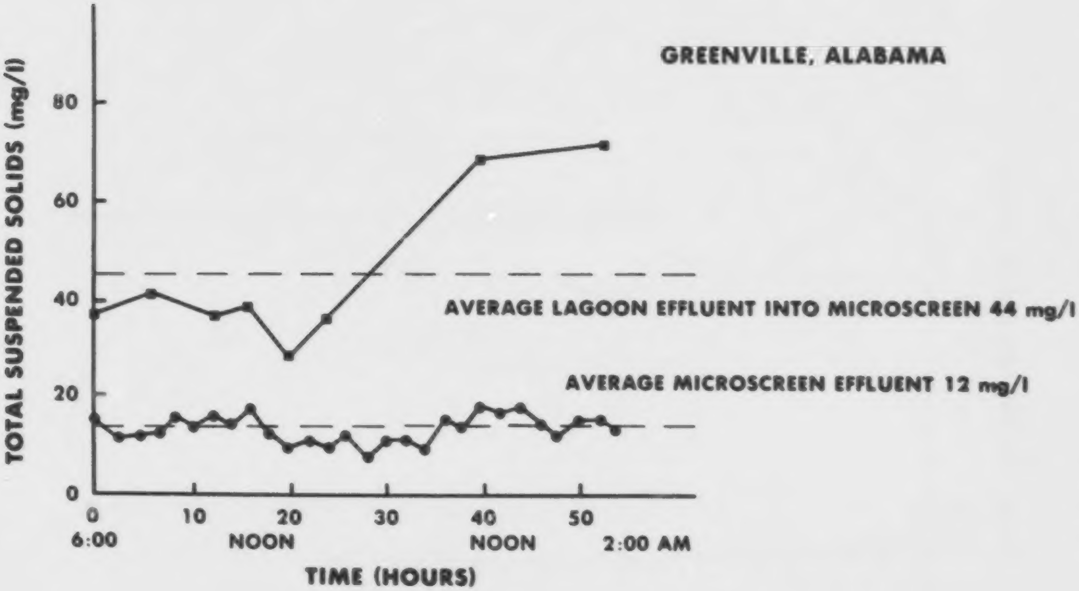

Figure 8. Microscreen influent and effluent suspended solids vs. time using 1 micron media.

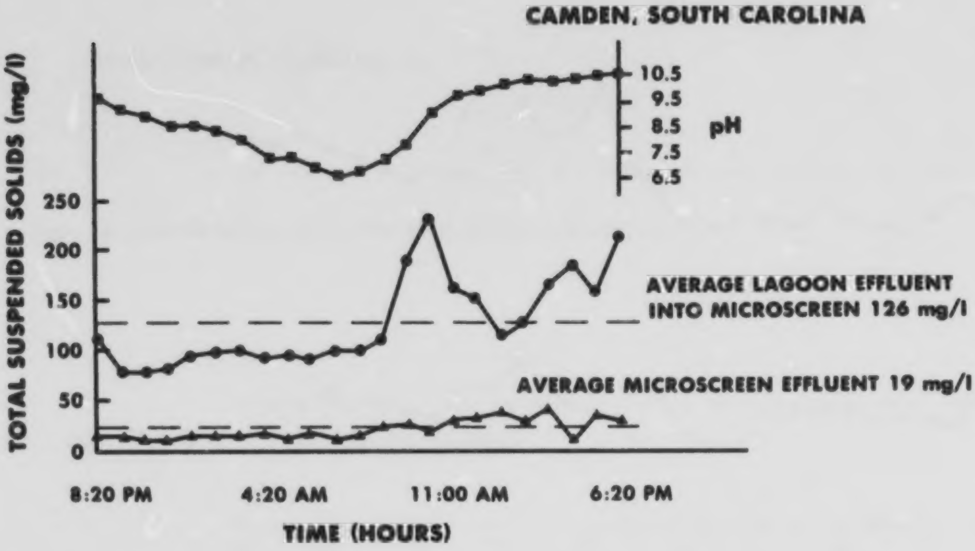

Figure 9. Microscreen influent and effluent suspended solids and pH vs. time using 1 micron media.

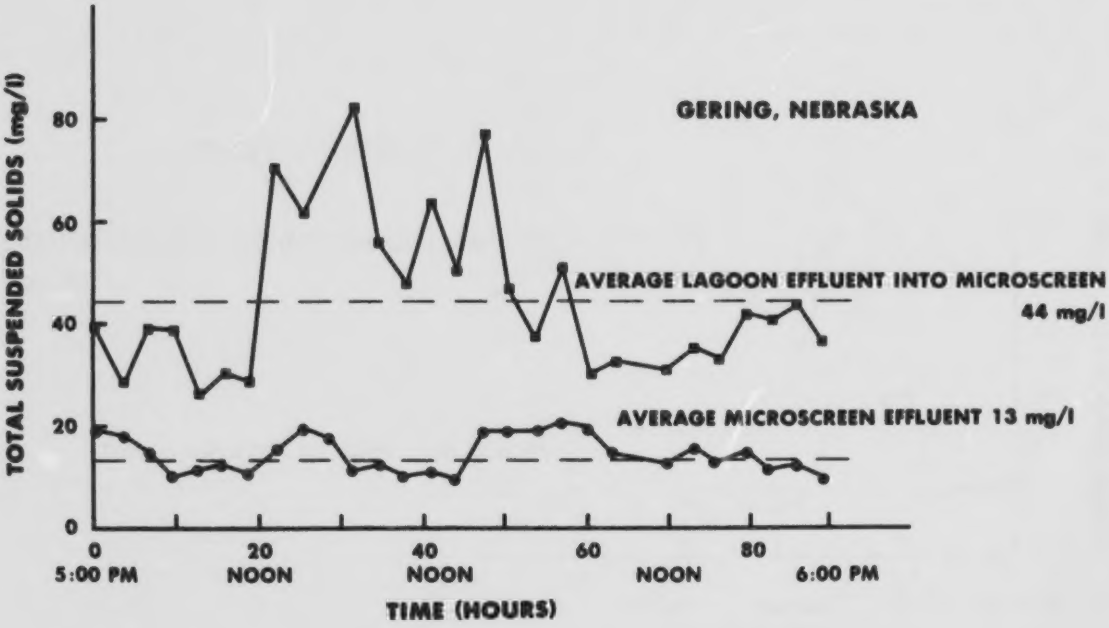

Figure 10. Microscreen influent and effluent solids vs. time using 1 micron media.

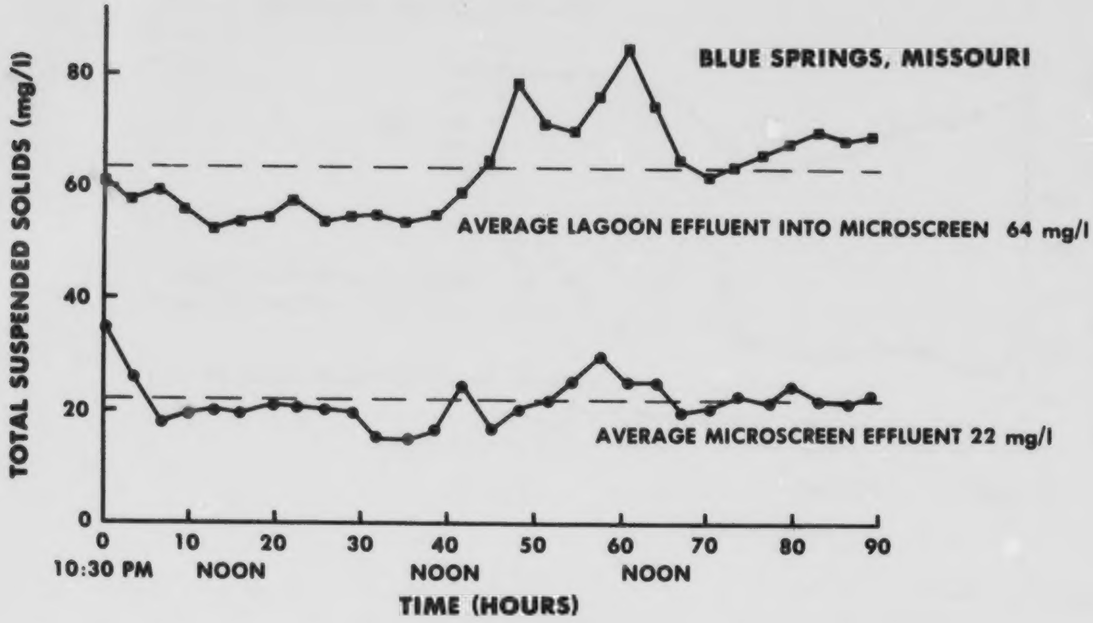

Figure 11. Microscreen influent and effluent solids vs. time using 1 micron media.

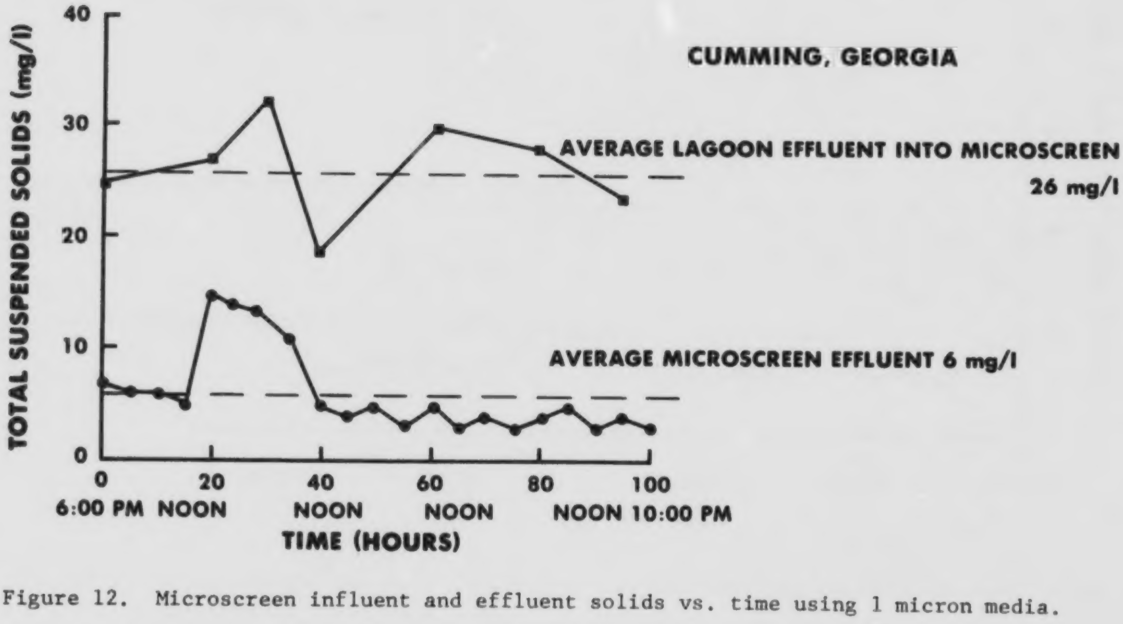

Figure 12. Microscreen influent and effluent solids vs. time using 1 micron media.

Table 3. Comparison of lagoon and microscreen effluents BOD/SS ratios vs. EPA lagoon standards.

Site	Actual Pond Discharge[a] (Microscreen influent)	Ratio	Required for EPA Standard[a]	Microscreen Effluent Actually Achieved
1	43/51	0.84	30/36	12/19
2	38/100	0.38	30/79	23/21
3	61/81	0.75	30/40	19/22
4	44/83	0.53	30/57	14/11
5	50/72	0.70	30/43	28/9
6	30/59	0.51	30/59	18/15

[a] 30 mg/l BOD currently required for lagoon effluent discharges less than 2 mgd.

Table 4. Summary of the average microscreen influent and effluent total suspended solids and T-BOD$_5$ (mg/l).

Figure Number	Lagoon Effluent (Microscreen Influent) TSS (mg/l)/T-BOD$_5$	Microscreen Effluent TSS (mg/l)/T-BOD$_5$
6	69/72	9/9
7	58/30	15/15
8	44/45	12/14
9	126/38	19/23
10	44/—	13/—
11	64/32	22/16
12	26/—	6/—

Table 5. Cost effective analysis for 1.7 mgd lagoon upgrade--microscreen option.

Capital Costs	
1. Microscreen equipment and chamber (installed)	$595,000
2. Laboratory, office, control room, etc.	$112,000
3. Chlorination	$ 35,000
4. Bar screens	$ 50,000
5. Aeration addition	$126,000
6. Piping and valves	$120,000
7. Pump station	$ 75,000
8. Grading and paving	$ 20,000
9. Electrical	$ 25,000
10. Miscellaneous	$ 60,000
Total Construction Estimate	$1,218,000
Engineering, legal fees, etc. (20%)	$ 243,000
Total Project Cost	$1,463,600
Say	$1,500,000

B. P. Barber and Associates, conducted pilot studies with DAF and microscreens at a 1.7 mg/l lagoon which needed total upgrading (8). Their findings are shown in Tables 5, 6, and 7. As can be seen, microscreening is by far more cost effective than DAF or building a new mechanical plant. What is interesting to note here, is the significantly higher cost for a system utilizing DAF for algae removal. The vast majority of the additional total cost is for operating cost due to the requirements for alum addition and the removal of the alum/algae sludge. With microscreens the algae are only returned at the inlet of

Table 6. Cost effective analysis for 1.7 mgd lagoon upgrade--microscreen option.

Operation and Maintenance		
1. Power		
6 drives drawing 5 hp	30 hp	
Backwash pump 5 hp	5 hp	
Aerators 6 at 15 hp	90 hp	
Pump station	10 hp	
	135 hp	
135 hp at $200		$27,000
2. Manpower		
1 full time at 12,000		
1 part time at 5,000		
		$17,000
3. Materials and Supplies		
Grid replacement (once/5 years) 160 at $30		$ 6,000
Motor, miscellaneous, etc.		$ 5,000
4. Chlorination		
2.1 cents/1000 gallons		$13,000
5. Miscellaneous		
Say		$ 3,000
		$71,000/Year

Table 7. Cost effective analysis summary 1.7 mgd lagoon upgrade.

A. Microscreen Option: Total Present Worth $1,500,000 (71,000 x 11.47)	$2,314,370
B. New Activated Sludge Plant (Orbal): Total Present Worth	$2,718,658
C. Dissolved Air Flotation (From Pilot Study): Total Present Worth	$3,765,000

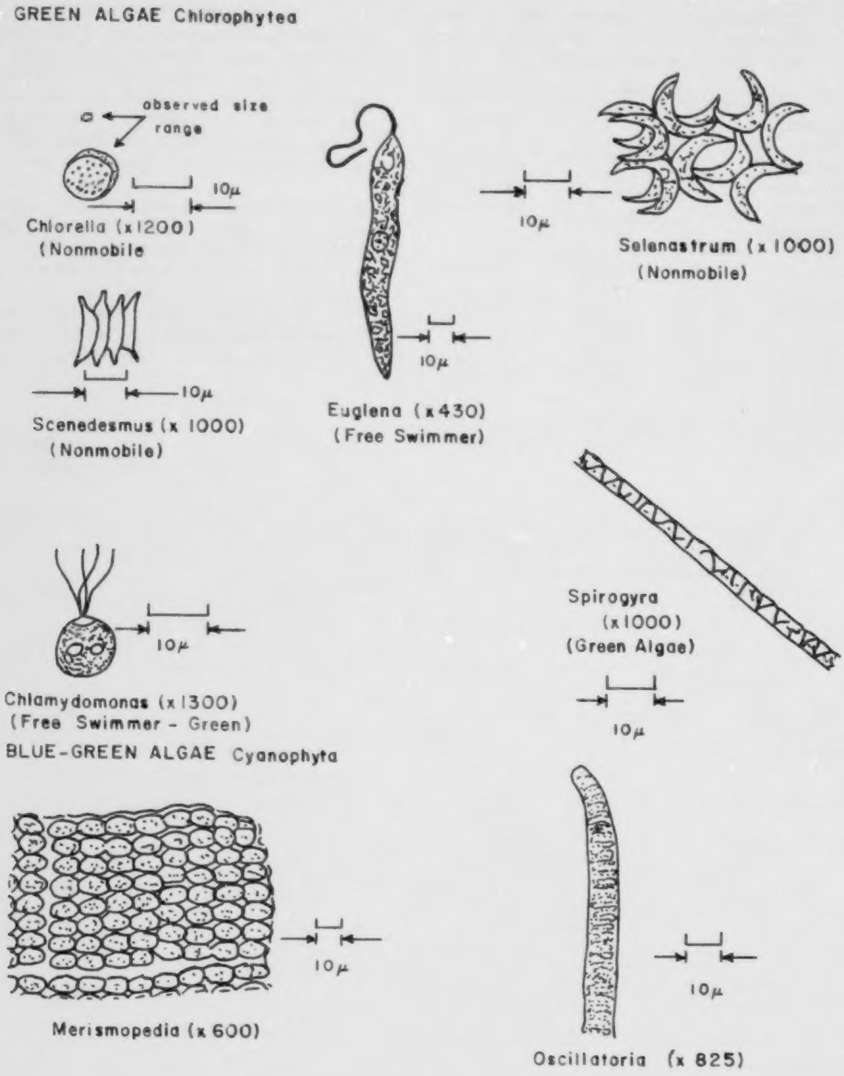
GREEN ALGAE Chlorophytea
observed size range
10µ
Chlorella (x1200)
(Nonmobile
10µ
Selenastrum (x 1000)
(Nonmobile)
10µ
Scenedesmus (x 1000)
(Nonmobile)
10µ
Euglena (x430)
(Free Swimmer)
10µ
Chlamydomonas (x1300)
(Free Swimmer - Green)
Spirogyra
(x1000)
(Green Algae)
10µ
BLUE-GREEN ALGAE Cyanophyta
10µ
Merismopedia (x 600)
10µ
Oscillatoria (x 825)

Figure 13. Typical algae species present in other oxidation ponds that may appear on a seasonal basis. (NOTE: 10 micron scale is shown in each case for the appropriate magnification).

the lagoon. This cost analysis tends to bear out the reason for the high operating costs at the Stockton installation which use DAF for algae removal.

CONCLUSIONS

1. Microscreening utilizating 1 micron polyester fabric effectively removes algae as demonstrated at eight lagoon sites and achieved a minimum of a 30/30 effluent or better at all sites researched.

2. Microscreening has been shown to require only minimum operating experience or expertise; thus, allowing for low operating expense. Other reported experience on this research (1,2) shows microscreening effectiveness to be independent of pH or volumes of algae (measured as suspended solids).

3. The financial analysis conducted by B. P. Barber and Associates (8) has shown microscreening to be very cost effective for the removal of algae.

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POND ISOLATION AND PHASE ISOLATION FOR CONTROL OF SUSPENDED SOLIDS CONCENTRATION IN SEWAGE OXIDATION POND EFFLUENTS

Ben L. Koopman, John R. Benemann, and W. J. Oswald*

INTRODUCTION

During the 30 years of lagoon research that has followed the earliest papers of the 1940s and 1950s (1, 2, 3) it has become amply evident that overflowing waste ponds, i.e., waste ponds which discharge effluents, should be comprised of more than one unit in series. Indeed, in climates having well defined seasons, a case has been made for a minimum of four waste ponds in series safeguarded from thermal short-circuiting influent to effluent by alternating low-level and high-level interpond transfer systems and by carefully locating their horizontal positions with regard to winds and currents (4). Once a multi-pond sequence is accepted as a design principle, the opportunity arises to design each pond in the series to optimize one or more stages in the series which are known to occur. We have previously used the term "phase isolation" to describe an environment for each distinct phase of the overall treatment process having special requirements (5).

As has been emphasized in several previous papers (6, 7), processes (e.g. phases) which can be selectively enhanced in specially designed ponds are grit and grease removal; primary sedimentation; methane fermentation of settled organic solids; biological oxidation of soluble organics; photosynthetic oxygen production; nutrient and toxicant removal; separation of algae; and disinfection. When ponds which perform these functions are placed in series, we refer to the entire process as "integrated ponding." The advantage of integrated ponding is that it is more effective and efficient, achieving higher treatment standards in smaller areas than standard facultative pond systems. This is due to the increased process rates achievable when design is optimized for each particular phase. At present design parameters for standard pond systems are not well defined (8) and designs for integrated ponding must to a considerable extent be based on standard pond design criteria. Thus the design criteria and formulae for integrated ponding systems are not yet perfected and require further R & D. Nevertheless, it is already possible to apply the integrated ponding concept to the design of wastewater treatment facilities. Each of the processes mentioned above can be enhanced through specific pond design details and each can contribute significantly to improvement in the final effluent from ponds. Not all the phases listed can or need be isolated from each other, and separation between phases is not perfect. Integrated ponding involves a combination of anaerobic, facultative, high-rate, settling, polishing, and holding ponds. In the simplest configuration of integrated ponds, the first of the above processes--primary sedimentation, grit and grease removal, methane fermentation and some oxidation--occur in the primary pond of the series which is a highly loaded facultative pond. Biological oxidation and photosynthetic oxygen production occur simultaneously in the second pond of the sequence, termed the algal growth or high-rate pond. The recirculation of oxygenated effluents with high pH from the algal growth pond to the plant influent may be used to condition the influent and improve the performance of the facultative pond. Removal through sedimentation of algae is best accomplished in a third stage of the process, the sedimentation phase, in an algal settling pond. A final polishing-holding pond in the sequence is recommended but not absolutely required to allow nutrient stripping, additional algal sedimentation, to permit further disinfection, and to provide storage for controlled discharge and/or land application of the treated waste.

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*Ben L. Koopman, John R. Benemann, and W. J. Oswald are respectively, Graduate Research Engineer, Associate Research Biochemist, and Professor of Sanitary Engineering and Public Health, University of California, Berkeley, California.

In this paper we present recent experience with the operation of an algal sedimentation pond at Woodland, California. This system was initiated by Hiatt, Engineer of the City of Woodland, who has reported (9) the production of high quality effluent by a system involving conventional facultative ponds followed by a draw and fill operated secondary pond. The function of the secondary, so called "phase isolation" pond was to accomplish removal of algae from the facultative effluent. It was not known by what mechanism this removal occurred although the isolation of the facultative effluent from further sewage inflow until the algae disappeared was deemed essential. The novelty of this treatment scheme led us to conduct a 1 year monitoring program of its application at Woodland, California, to treat 1 mgd of sewage. This study has now been extended, but is nearly completed and preliminary results can be reported. To avoid confusion we have suggested that the term "pond isolation" be applied to the removal of algae from facultative pond effluents on a batch basis in an isolated secondary pond.

Results from Woodland are compared with those obtained in integrated pond systems. Recent experiences at our laboratory at Richmond, California, in the removal of algae by microstraining are also discussed.

POND ISOLATION STUDIES AT WOODLAND

A still ongoing study of the pond isolation process has been carried out by our group under sponsorship and in conjunction with the City of Woodland and the California Water Resources Control Board during 1977 and 1978 using existing ponds of the City of Woodland. Pertinent details of that study drawn from our reports to the City of Woodland and the California Water Resources Control Board (10) are outlined here.

Climate

The City of Woodland is located in North Central California. This area has warm, dry summers and moderately warm, wet winters. Weekly averages for insolation, air temperature, and wind movement are presented in Figure 1. Data for rainfall, evaporation, and net evaporation are presented in Table 1. Daily isolation averaged greater than 500 langleys between April and September, but rarely rose above 200 langleys in December and January. Net pan evaporation was greatest in June and July (exeeding 30 cm per month) and averaged over 25 cm per month between April and October. Significant amounts of rain fell in November and continued intermittently throughout December and January.

Table 1. Rainfall, evaporation, and net evaporation in Woodland area.[a]

Month	Precipitation cm	Evaporation cm	Net Evaporation cm
April 1977	T	24.6	24.6
May	2.8	19.0	16.2
June	T	32.1	32.1
July	T	33.2	33.2
Aug.	-0-	27.9	27.9
Sept.	1.8	22.1	20.3
Oct.	0.6	16.2	15.6
Nov.	7.0	10.0	3.0
Dec.	10.1	3.8	<6.3>
Jan. 1978	24.7	2.5	<22.2>

[a] U.C. Davis Meteorological Station.

As indicated in Figure 1, the Woodland area was moderately windy during the entire investigation and especially so in April 1977 and January 1978 when average wind movements of over 250 km per day were recorded. Average maximum air temperatures (Figure 1) exceeded 30°C from June into October. Average minimum air temperatures were usually about 15°C below the maximums.

Water Supply and Wastewater Composition

The City of Woodland obtains its water from deep wells. A typical chemical analysis of the domestic water supply is given in Table 2.

Woodland's wastewater is principally domestic in origin. Industrial wastewaters are treated in separate facilities from the domestic wastewater. Observations of the domestic sewage during the canning season indicated, however, that some tomato processing wastes were discharged into the domestic sewer. A typical analysis of Woodland's domestic wastewater is given in Table 3.

Ponding System

The Woodland facultative-isolation ponding system consists of six, square-shaped, 4.9 ha ponds. These ponds were excavated from alkali-type soil and were not lined. The layout and flow schemes for these ponds are shown in Figure 2.

Ponds 1, 2, and 3 were about 2.7 m in depth and Pond 6 was about 1.8 m deep. Four of the ponds, numbers 1, 2, 3, and 6 were loaded with raw sewage. Sewage was introduced by way of bottom inlets located about 40 m from the closest pond levees. Pond 5 was normally operated as the isolation pond. After being partially drained, it could be filled again with effluents from Ponds 1, 2, and 6. Because of hydraulic constraints, Pond 3 could not discharge to Pond 5 and, therefore, was usually allowed to overflow continuously into a drainage channel. During a typical discharge period, about the upper 1/8 m of water depth could be withdrawn from each facultative pond. The isolation pond typically reached a depth of 1.5 m after being filled.

Once in the isolation pond, the facultative effluent was held without further sewage inflows until a marked decline in algal concentration occurred. When the TSS concentration in the isolation pond dropped below 30 mg/l or appeared to have reached a minimum, the supernatant (usually 0.4-0.5 m of depth) was discharged. The supernatant was withdrawn via a concrete pipe located at the NW corner of the pond. The draining period required 3 to 4 days. About 1.1 m of water depth

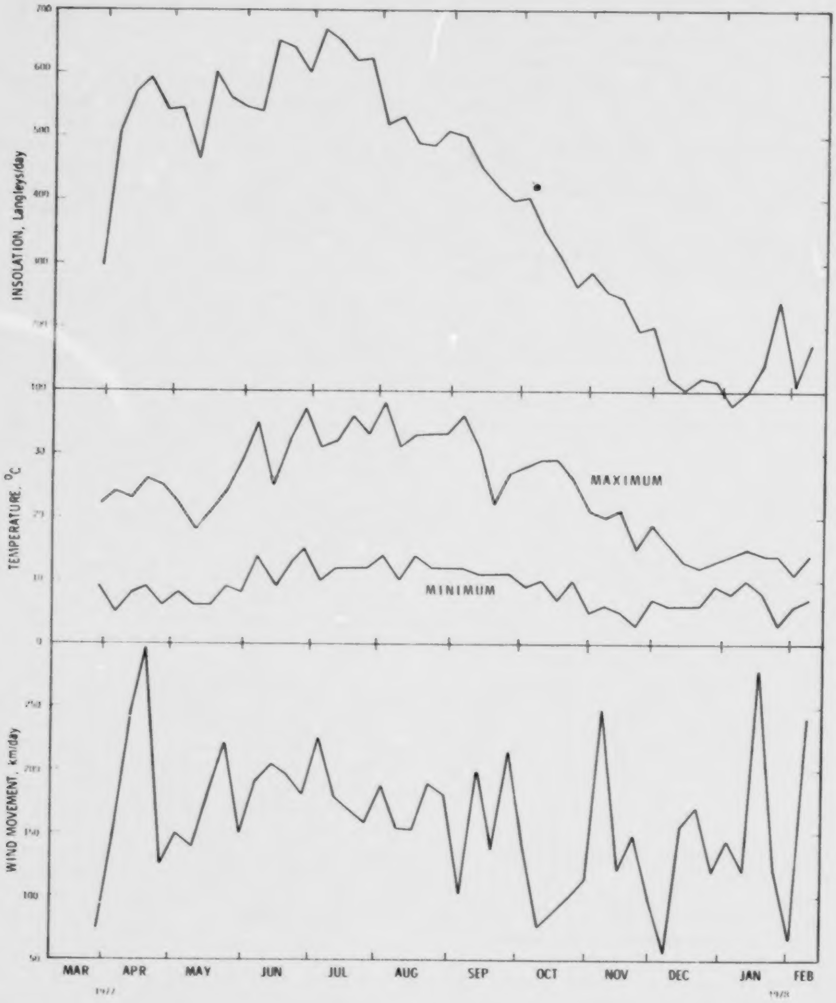

Figure 1. Weekly average wind movement, air temperature, and insolation in Woodland area.

remained in the isolation pond after discharge because of the vertical placement of the pipe. When the drain cycle was completed, flashboard risers connecting Ponds 2 and 6 to the isolation pond were opened. Effluent from Pond 1 first flowed into Pond 6 before reaching the isolation pond. Water surface elevations between the facultative and isolation ponds required about 2 days to equalize and, thus, complete a fill cycle.

Table 2. Typical chemical analysis Woodland domestic water supply (6).

Total solids	368 mg/l
Hardness	248
Bicarbonate	258
Alkalinity	258
Ca	41.5
Mg	30.5
Fe	<0.02
Mn	<0.02
Na	50
K	2.0
Chlorides	58
Sulfates	34
Fluorides	0.5
Nitrate-Nitrite	3
Specific Conductance	484 micromhos/cm
pH	8.2
As	<0.006 mg/l
B	1
Cu	<0.005
Pb	<0.005
Se	0.005
Zn	0.008
Hg	<0.002
Color	<5 units
Odor	None
Temperature	65°F
Turbidity	<1 unit

Table 3. Typical analysis--Woodland domestic wastewater.

COD	
total	364 mg/l
filtered	137
particulate	227
BOD_5	184
Nitrogen	
total	39.1
ammonia	26.6
nitrate + nitrite	0.1
organic	12.4
Total Phosphate	10.6
Total Iron	2.2

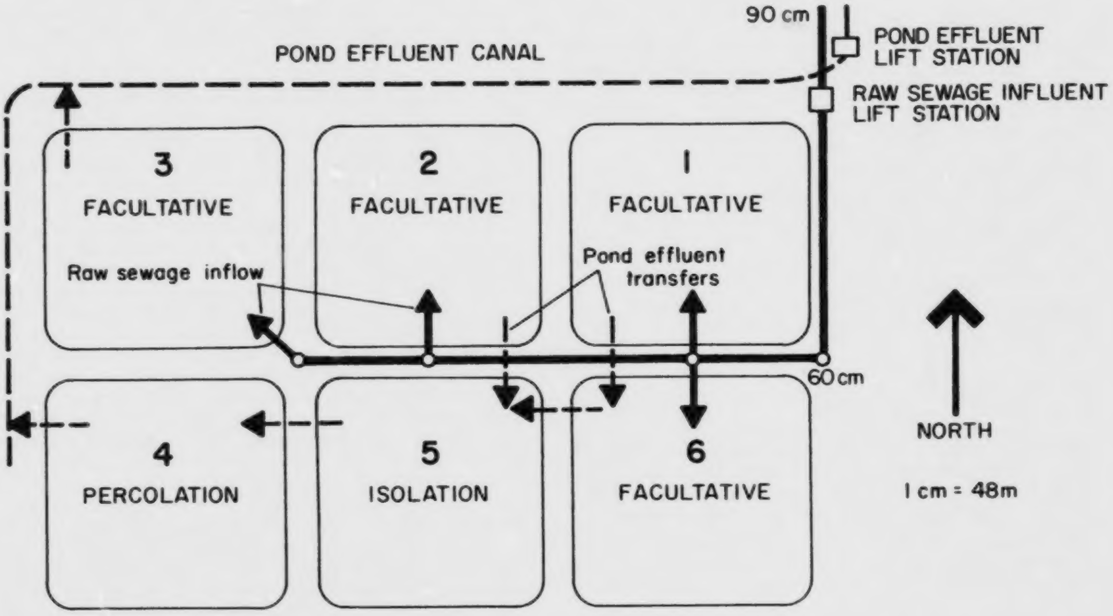
90 cm
POND EFFLUENT LIFT STATION
POND EFFLUENT CANAL
RAW SEWAGE INFLUENT LIFT STATION
3
FACULTATIVE
2
FACULTATIVE
1
FACULTATIVE
Raw sewage inflow
Pond effluent transfers
60 cm
4
PERCOLATION
5
ISOLATION
6
FACULTATIVE
NORTH
1 cm = 48m

Figure 2. Layout and present flow scheme for Woodland strong ponds. Redrawn from (11).

Sampling Schedule and Locations

Raw Sewage--

The raw sewage was sampled at the lift station after being screened and pumped. The lift station was located about 3 km from the pond system. Three distinct samples were taken each day (morning, noon, and afternoon), and these were composited prior to analysis. Two or three such composite samples were obtained each week.

The flowrate of sewage was measured immediately before reaching the facultative ponds. The differential pressure across a Venturi section was sensed by a mechanical unit[1] which recorded the instantaneous flowrate as well as totalizing daily flows.

Facultative Pond Effluents--

The effluents from Ponds 1 and 6, which reached the isolation pond via a single transfer structure, were sampled using an Isco automatic composite-sampler, Model 1580. The duration of compositing was limited to about 8 hours daily because of the possibility of vandalism. Effluent from Pond 2 was sampled and composited manually. Four individual samples were taken the first day of overflow and three on each day thereafter, over a period of 8 hours.

The volume of effluent represented by each composite sample was calculated from the daily changes in water surface elevations of the ponds sampled. For a 4.9 ha pond, each centimeter of depth represented about 490 cu m of effluent.

Isolation Pond Effluent--

Effluent samples from the isolation pond were composited using the Isco automatic sampler. The sampling period was approximately 8 hours daily. As noted above, the volume of effluent represented by each day's sample was calculated from the daily change in pond elevation.

Isolation Pond *In Situ* or Offshore Sampling--

Samples of the isolation pond during non-draining periods were taken either at the geometric center of the pond or from the NW corner of the pond near the drain pipe. *In situ* samples at the center of the isolation pond were obtained at 0.30 m intervals using a Van Dorn bottle and composited to give a water column average. Offshore samples were taken at a depth of 0.15 m about one-half meter from the pond's edge.

Facultative Ponds: Loading and Effluent Compositions

Raw Sewage Loading Rates--

The BOD_5, COD, and total nitrogen content of the sewage influent to the facultative ponds is given in Table 4. BOD_5 averaged 238 mg/l over the reporting period, a value considerably higher than the average BOD_5 of 147 mg/l reported in 1975 (11). The high levels of BOD_5 recorded in 1977 were also characteristic of wastewaters from other California cities during the drought of that year. Because of impending water shortages, water conservation efforts substantially reduced normal water consumption in the City of Woodland.

The highest monthly averages for BOD_5 were recorded in May and August when values of 355 mg/l and 305 mg/l, respectively, were found. The high August BOD_5 apparently reflected the contribution of tomato wastes to the domestic sewage as remnants of tomatoes were observed in the sewage during this period.

Total nitrogen averaged 34.9 mg/l, also a typical value for domestic sewage. The nitrogen concentrations were quite uniform except for minimums of 27.5 mg/l and 28.5 mg/l observed in July and December.

Average total inflow to the facultative ponds was 4,400 cu m per day, the same as recorded in 1975. However, for the three-month period--July-September--the average flow dropped to 3,750 cu m daily. This drop in the inflow rate resulted in very small volumes of overflow from the facultative ponds because evaporation and percolation were quite high during this period. In order to restore a greater volume of facultative pond effluent, additional sewage from the lift station was diverted to the facultative ponds beginning in October. In response, the inflow rate increased to an average of 4,800 cu m per day between October 1977 and January 1978.

Also given in Table 4 are hydraulic, BOD_5, and total nitrogen loading rates to the facultative ponds. These rates were calculated by dividing the respective mass loading rates by the total area of the four facultative ponds receiving sewage. The hydraulic loading rate averaged 2.2 cm per day. However, during the months of July, August, and September, the loading rate dropped to only 1.9

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[1] Honeywell Corp., Model No. Y222(E)1.

Table 4. Raw sewage loading to Woodland facultative ponds--April 1977 to January 1978.

Month	Flow m^3/day	COD mg/l	BOD_5 mg/l	Total Nitrogen mg/l	Overall Loading Rates Hydraulic cm/day	BOD_5 kg/ha/day	Nitrogen kg/ha/day
April	3,700	479	246	38.9	1.9	47	7.4
May	4,200	696	355	36.0	2.2	77	7.8
June	5,300	393	202	37.0	2.7	55	10.1
July	3,000	320	165	27.5	1.5	25	4.2
Aug.	4,500	571	305	36.5	2.3	71	8.4
Sept.	3,750	472	257	35.2	1.9	50	6.8
Oct.	4,800	444	255	38.5	2.5	63	9.5
Nov.	5,000	440	237	40.1	2.6	61	10.3
Dec.	4,700	346	181	28.5	2.4	44	6.9
Jan. 1978	4,700	339	193	31.4	2.4	47	7.6
Ave.	4,400	446	238	34.9	2.2	54	7.9

cm per day. Evaporation alone during this period accounted for 0.7 cm per day,[2] thus reducing the effective loading rate to 1.2 cm per day. Percolation undoubtedly also removed a significant amount of water each day. The net result was that the quantity of effluent discharged was insufficient to keep the isolation pond adequately diluted. Beginning in October, the hydraulic loading rate was increased. Between October and January 1978, the average hydraulic loading was 2.5 cm per day.

The average BOD_5 loading rate was 54 kg per ha per day, a value considerably higher than the rate of 44 kg per ha per day reported for 1975. This increase was due solely to an increase in BOD_5 concentration as the flow rates for both years were similar.

Peak BOD_5 loading rates of over 70 kg per ha per day were recorded in May and August. Although these loadings were considerably greater than the average rate of 34 kg per ha per day allowed for design in most states (12), there were no odor nuisances or occurrences of floating sludge observed in the ponds.

The average total nitrogen loading rate to the ponds was 7.9 kg per ha per day. Peak monthly values of 10.1 and 10.3 kg per ha per day occurred in June and November, respectively.

Performance of Primary Ponds--

The intermittent nature of the facultative pond overflows facilitated characterization of these wastes. Usually only 2 days of sampling were required to monitor the discharge resulting from 2 to 4 weeks of pond operation. The compositions of these overflows are given in Table 5.

Overflow volumes--Volumes of overflow were calculated using observed changes in pond depth. The processes of evaporation, percolation, and sewage inflow continued to affect pond depths during the discharge periods, but their combined effects were insignificant relative to the larger changes in depth caused by discharge. The combined losses in facultative pond depths from discharge were found to be within 10 percent, on the average, of the corresponding rise in depth of the isolation pond.

Between June and August an average of 20,200 cu m of effluent were discharged to the isolation pond each month. As no overflows of the isolation pond occurred during this period, it is apparent that the combined effects of evaporation and percolation were responsible for removing approximately 1.4 cm of water per day. Considering the average pan evaporation rate for this period (0.90 cm per day) and a pan coefficient of 0.75, about 50 percent of this removal can be attributed to evaporation, with the rest due to percolation.

The 57,600 cu m of effluent discharged between September 27 and October 4 was due to two factors: (1) four facultative ponds, rather than three, contributed effluent and (2) Pond 4,

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[2] By application of a pan coefficient of 0.75 to the average net pan evaporation during July-September of 0.90 cm per day.

Table 5. Compositions of Woodland facultative ponds' overflows.

Overflow Period	Source Ponds No.	Volume	Suspended Solids			Chlorophyll $\underline{a}$	BOD_5	COD		Nitrogen			
			Total[b]	Volatile[c]									
		m^3	mg/l	Algal mg/l	Grazer mg/l	µg/l	mg/l	Total mg/l	Filtered mg/l	NH_3+ NH_4^+ mg/l	NO_3^-+ NO_2^- mg/l	Organic mg/l	Total mg/l
April 18-20 1977	1&6	--	139	117		1460	--	206	52	0.2	--	12.0	12.2
	2	--	84	70		577	--	178	64	7.5	--	9.4	16.9
	Comp	--	111	93		1020	--	192	58	3.8	--	10.7	14.5
May 17-18	1&6	--	68	59		631	--	171	59	0.8	--	8.7	9.5
	2	--	90	78		759	--	109	28	0.3	--	9.4	9.7
	Comp	--	77	67		682	--	146	47	0.6	--	9.0	9.6
June 6-7	1&6	--	66	46		548	--	183	77	0.2	--	6.5	6.7
	2	--	113	96		866	--	190	37	0.1	--	9.0	9.1
	Comp	--	85	66		675	--	186	61	0.2	--	7.5	7.7
June 27-28	1&6	13,800	76	60	--	424	85	--	92	0.2	--	6.5	6.7
	2	8,400	100	85	--	1120	46	--	60	0.3	--	10.6	10.9
	Comp	22,200	85	69	--	690	70	--	80	0.3	--	8.3	8.6
July 26-27	1&6	9,500	75	62	--	403	--	--	81	0.2	--	5.6	5.8
	2	7,400	113	96	--	1020	--	--	54	0.3	--	8.4	8.6
	Comp	16,900	92	77	--	674	--	--	69	0.2	--	6.8	7.0
Aug. 22-23	1&6	12,300	111	75	11	562	51	--	72	0.6	T	10.4	11.0
	2	9,300	129	87	16	1004	39	--	59	1.9	T	12.7	14.6
	Comp	21,600	118	80	13	752	46	--	66	1.2	T	11.4	12.6
Sept. 27[a] -Oct. 4	1,2,3 &6	57,600	133	79	13	891	74	--	83	2.5	0.1	9.6	12.2
Oct. 24-26[a]	1,2,3 &6	24,600	161	99	26	1070	--	--	67	3.6	0.1	10.1	13.8
Nov. 28-29	1&6	18,000	87	49	16	451	--	125	--	0.2	0.7	6.9	7.8
	2	17,200	82	65	3	779	--	146	--	1.5	0.5	10.2	12.2
	Comp	35,200	84	57	10	612	--	135	--	0.8	0.6	8.5	9.9
Dec. 21-22	1&6	20,100	58	46	2	459	26	124	70	0.1	1.3	7.2	8.6
Jan. 17-18	1&6	25,300	33	17	1	97	18	71	39	0.8	1.0	3.1	4.9
Feb. 8 1978	1&6	14,500	35	20	1	195	18	73	24	1.8	1.1	2.8	5.7
	2	10,100	37	29	-0-	344	22	79	46	3.3	0.9	3.8	8.0
	Comp	24,600	36	24	1	256	20	75	33	2.4	1.0	3.2	6.6

[a] Overflows discharged to temporary isolation pond.

[b] Grazers not included in TSS for June 27-28 and July 26-27 overflows.

[c] Algae and grazers separated with 150-µm or 243-µm mesh straining cloth.

[d] NO_3^- and NO_2^- not included in total-N April-July.

which was completely empty before, was filled instead of Pond 5, thus allowing the facultative ponds to be drawn lower than normal. Again, during October 24-26, all four facultative ponds were discharged into Pond 4.

Beginning in November, overflows were again routed into Pond 5. In December and January only Ponds 1 and 6 were allowed to discharge into Pond 5 while in November and February, overflows from Ponds 1, 2, and 6 were incorporated into the fill stream.

Suspended solids--The suspended solids contents of the facultative pond overflows are shown in Figure 3. In this figure the total bar height represents the total suspended solids (TSS) and the height of the shaded area gives the volatile suspended solids. Beginning with the October outflow, the VSS is divided between algal and grazer biomass.

Differentiation between algal and grazer biomass--Significant populations of grazer organisms including water fleas (Daphnia), copepods (Cyclops), and rotifers (Brachionus) were often observed in the ponds.

Initially, no effort was made to separate the algal grazer biomass. Beginning June 27, the effluent samples were prestrained with 243 µm mesh fabric before making dry weight determinations. Any visible Oscillatoria filaments retained by the fabric were rinsed through with previously strained effluent. The fabric retained Daphnia and most copepods but allowed rotifers to pass. The procedure was continued through the June 27-28 and July 26-27 overflows.

In order to quantify the grazer biomass, additional dry weight determinations were made starting August 22 on the biomass retained by the fabric. Debris such as twigs or insects were excluded from these analyses. Dry weights for the prestrained effluent and the grazer biomass were added to give the "total suspended solids" values reported in Table 5. The volatile or ash-free dry weights of the prestrained effluent were reported as "algal suspended solids" and ash-free dry weights of grazer biomass as "grazer suspended solids."

After September 30, fabric with smaller, 150 µm openings was substituted for the 243 µm mesh fabric. This enabled many of the larger rotifers, in addition to Daphnia and copepods, to be included in the grazer biomass determinations.

Total suspended solids--None of the overflows from facultative ponds contained less than 30 mg TSS per liter. Two overflows (January and February)

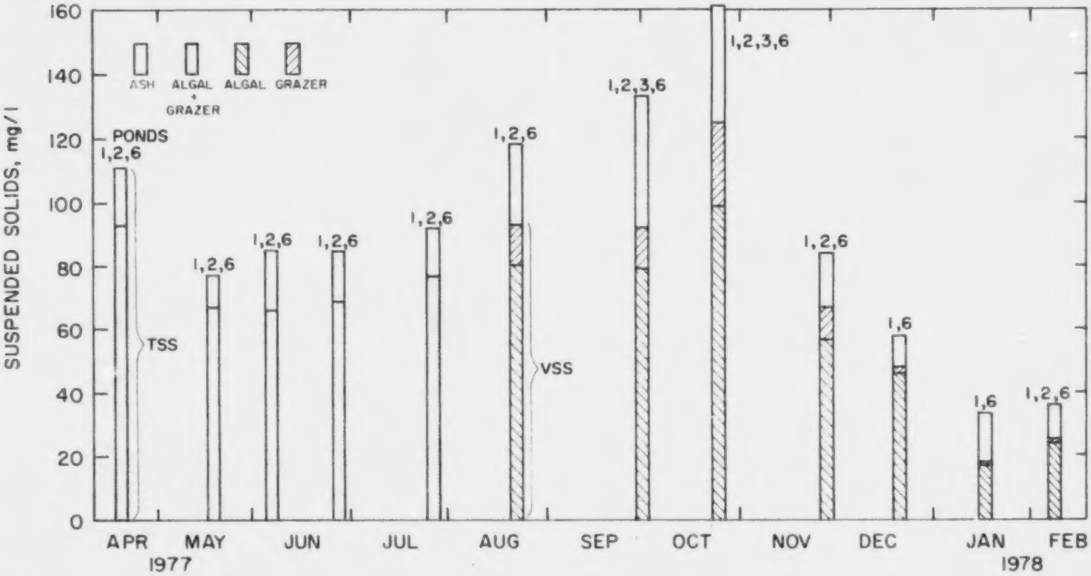

Figure 3. Total, volatile, algal, and grazer suspended solids in Woodland facultative pond overflows.

contained no more than 40 mg/l. Effluent TSS levels were greater than 70 mg/l in most cases and equaled or exceeded 100 mg/l in April, August, September, and October discharges.

Volatile suspended solids--On the average 77 percent of the total suspended solids consisted of volatile matter. This proportion varied somewhat between discharges and appeared to depend, in part, on wind conditions. Samples taken on windy days contained finely divided, greyish material probably comprised of resuspended sediments. During stormy weather in January and February, the average effluent-suspended solids were only 62 percent volatile.

Algal and grazer volatile suspended solids--A significant fraction, 11 percent, of the average VSS consisted of grazer biomass. Concentrations reached 26 mg/l in October when grazers made up 21 percent of the volatile suspended solids. Relatively small grazer populations, averaging less than 2 mg/l, were observed in December, January, and February, and comprised 5 percent of the VSS during that period. Algal biomass in the January and February overflows was less than 30 mg/l. All other discharges contained at least 46 mg/l algal biomass with a peak of 99 mg/l occurring in October.

Chlorophyll a--Chlorophyll *a* was used as another index of algal biomass. It is not contained by the non-photosynthetic components of pond ecosystems but is present in all the pigmented algae. The chlorophyll *a* content of algal cells is subject to variation, depending on light intensity, nitrogen availability, cell age, and other physiological factors. However, the median of a large number of samples is often found to be about 1 percent of the ashfree dry weight (13).

Chlorophyll *a* correlated linearly with both the algal suspended solids ($r^2 = 1.0$) and the total suspended solids ($r^2 = 0.9$).

Biochemical oxygen demand (BOD)--The BOD_5 measurements were conducted in the dark to prevent photosynthetic oxygen production. In BOD_5 tests of Woodland's pond effluents, it was observed that algal cells and grazer organisms usually survived the incubation period. Therefore, mainly the respiratory oxygen requirements for these organisms were measured. Relatively few BOD_5 assays were made on facultative pond overflows. Discharges in December, January, and February contained less than 30 mg/l. The average BOD_5 of the other discharges was 63 mg/l.

Chemical oxygen demand (COD)--COD tests were run on filtered samples of pond effluent in order to provide an estimate of their soluble organic strength. The difference between filtered and unfiltered (total) COD gave the particulate COD, i.e., the COD contributed by algal biomass. Filtered COD was lowest in the January and February overflows. The average for these months was 36 mg/l. Filtered COD was 80 mg/l in the June 27-28 discharge and exceeded this value in September. The average for all other overflows was 65 mg/l. Total COD measurements are incomplete. Using those available, the particulate COD is found to have ranged between 134 mg/l in April and 32 mg/l in January.

Nitrogen--The total nitrogen was determined by summing ammonia-N, nitrite plus nitrate-N and organic N. The highest concentrations of effluent, total N, exceeding 10 mg/l, occurred in April and August-October. Only the January discharge contained less than 5 mg N per liter.

The principal constituent of the total nitrogen concentration was always the organically bound nitrogen as determined by the Kjeldahl test. Organic nitrogen accounted for 75 percent of the average total N value in those analyses for which all constituents were determined.

Ammonia was the next most important nitrogen species, occurring at concentrations of 0.2 to 3.0 mg/l. In every discharge except that for January, ammonia exceeded the sum of nitrite and nitrate on a nitrogen basis. Ammonia dropped below 1 mg/l between May and July and again from November through January. Nitrate and nitrite were not measured before August. Their sum was below 1 mg/l until December and remained near 1 mg/l thereafter.

Algal composition of facultative pond overflows--One of the two algae predominant in most overflows was *Oscillatoria*. *Oscillatoria* is a filamentous, blue-green alga commonly tychoplanktonic in nature. Along with many other blue-greens, *Oscillatoria* have the ability to regulate their buoyancy through the use of intracellular gas vesicles and thus can migrate vertically in the water column. The species observed at Woodland occurred in trichomes about 6 µm in diameter and ranging from less than 50 µm to several millimeters in length. When ponds at Woodland were rich in *Oscillatoria*, they appeared turquoise in color and often exhibited surface scums in downwind corners.

The second predominant type of algae has not yet been identified. They are grass green in color, approximately spherical in shape, and 3 µm in diameter. Because of their small size and lack of a distinctive, cup-shaped chloroplast, they were not believed to be Chlorella.

The algal compositions of individual pond discharges differed significantly. Oscillatoria made up a 41 percent or more of the total algal biovolume concentrations in Pond 2 effluents, whereas effluents from Ponds 1 and 6 contained at the most 23 percent Oscillatoria. The population of the unidentified green organism in Ponds 1 and 6 made up an average of 57 percent of the algal biovolume while these same algae never accounted for more than 26 percent of Pond 2 effluent algal biovolumes.

A detailed account of the algal species throughout the study is included in the project reports (10).

Isolation Ponds

Effluent Suspended Solids and Chlorophyll--

A graphic overview of the algal suspended solids (algal SS) and chlorophyll a found in the isolation ponds as a function of time is given in Figure 4. Algal SS, as noted previously, refers to the volatile suspended solids exclusive of grazer biomass.

The cyclic behavior exhibited in April, May, and early June corresponds closely to Hiatt's observations (9). During these cycles, the in situ algal SS and chlorophyll always fell from the maxima created by influxes of facultative pond effluent (fills). After the June 27 fill, however, the algal density rose and, despite some fluctuations, continued to increase throughout July and August and much of September.

A second anomaly occurred coincident with the growth of algae in the isolation pond: The increments of depth added by the June 27, July, and August fills were removed so fast by evaporation and percolation that discharges became impossible, the water levels having dropped below the elevation of the drain pipe invert before it was time for the next fill.

As October approached and no decrease in algal density or chance for discharge were in sight, it was decided to drastically modify pond operation. The exact strategy of modification was much debated, however. One school of thought held that algal sediments in Pond 5, having built up during previous years of operation, had begun recyling nutrients which sustained algal growth. Thus, it was reasoned that a previously unused pond should be substituted for the "aged" isolation pond. A second hypothesis presumed that, because there was no dilution through the isolation pond, algal species tolerant of low nutrient concentrations were able to colonize the pond in spite of their slow growth rates. If this hypothesis were true, then periodic discharges from the isolation pond would be needed to dilute out the nuisance-algae and restore proper function. The hypotheses were not, of course, mutually exclusive.

As a compromise, actions suggested by both hypotheses were taken. First, hydraulic loading rates to the facultative ponds feeding the isolation pond were increased in order to provide more facultative effluent for fills. Secondly, the September and October facultative overflows were directed into Pond 4, which had previously been unused, in order to determine if the absence of algal sediments would prevent algal growth during isolation.

Use of the "new" isolation pond did not restore effective algal removals or prevent additional algal growth. In fact, algae thrived in Pond 4, reaching algal SS and chlorophyll a levels of 94 mg/l and 1020 µg/l, respectively.

In November the facultative overflows were redirected to Pond 5. Subsequently, Pond 5 algal density began a steady decline which continued through the December isolation cycle. By the end of January, the algal density had apparently "bottomed out" at 5 mg algal SS per liter and 10 µg chlorophyll a per liter.

Compositions of Effluent Components--

The compositions of major components of interest in the effluents from the isolation ponds are given in Table 6.

Volumes--The volumes of effluent in the June and December overflows were relatively small, averaging 12,600 cu m. The June overflow was diminished in volume by the high evaporation and percolation rates occurring at that time. Between 27 June and 23 August, 60,700 cu m of facultative effluent was discharged into the isolation pond. This entire volume of water was lost through evaporation and percolation. A substantial portion of the November facultative overflow (58 percent) also disappeared during its retention in the isolation pond. The January and February isolation pond effluents averaged 24,000 cu m. The

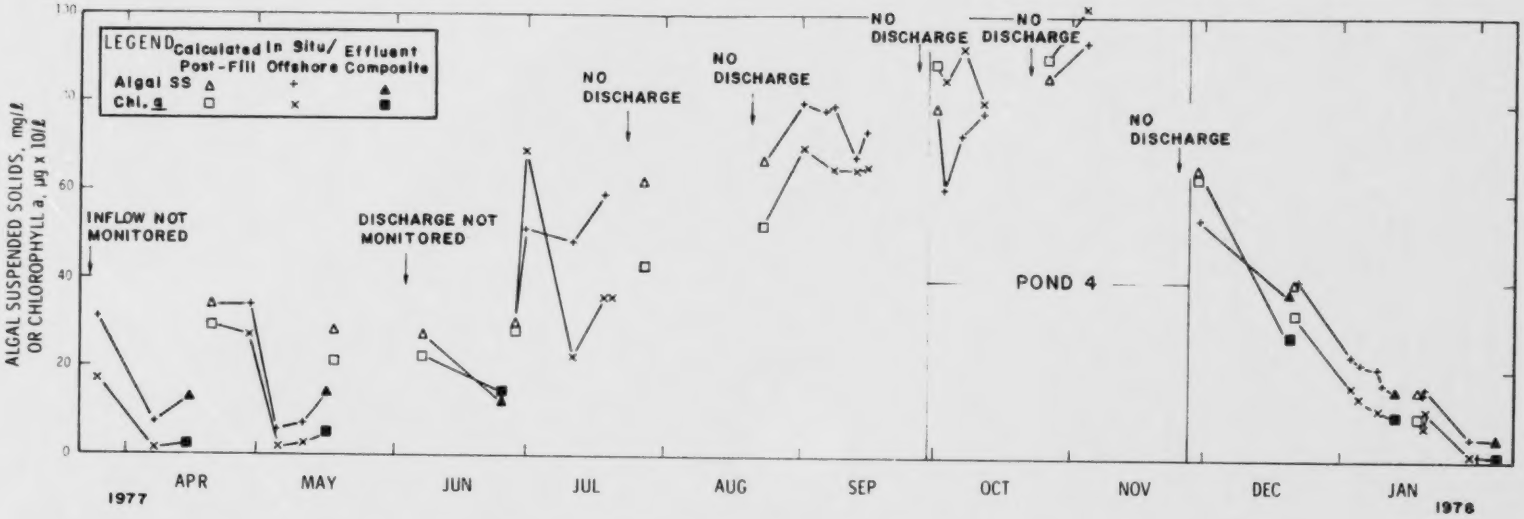

Figure 4. Algal suspended solids and chlorophyll a in isolation ponds, April 1977-January 1978.

Table 6. Compositions of Woodland isolation pond overflows.

Drain Period	Volume m^3	Suspended Solids				Chlorophyll a	BOD_5	COD		Nitrogen			
			Volatile[a]							NH_3+	NO_3^-+		
		Total mg/l	Algal mg/l		Grazer mg/l	µg/l	mg/l	Total mg/l	Filtered mg/l	NH_4^+ mg/l	NO_2^- mg/l	Organic mg/l	Total[b] mg/l
April 12-15 1977	--	42	--	13	--	27	--	78	60	1.3	--	2.4	3.7
May 12-16 1977	--	34	--	14	--	47	--	44	--	0.3	--	3.6	3.9
June 22-25 1977	10,500	28	12		3	114	--	77	--	0.4	--	4.1	4.5
Dec. 18-20 1977	14,800	64	38		6	281	10	117	68	1.1	1.3	5.6	8.0
Jan. 11-13 1978	25,300	31	16		3	100	13	59	46	0.7	1.2	3.5	5.4
Feb. 3-5 1978	22,700	36	5		6	8	5	49	35	1.1	1.1	1.8	4.0

[a] Algae and grazers separated with 150 µm or 243 µm mesh straining cloth.

[b] NO_3^- and NO_2^--N not included in Total-N April and May.

substantial rains during January and February more than offset percolation as the volume of effluent from the isolation pond exceeded its influent by 6 percent.

Suspended solids, total suspended solids--Effluent TSS averaged 39 mg/l over the report period. The minimum TSS, 28 mg/l, was observed in the June 22-25 effluent while the maximum, 64 mg/l, occurred in December. Except for December, all effluents contained 42 mg TSS per liter or less. If December were excluded, the average effluent TSS would be 34 mg/l.

Ash--It can be seen from Figure 5 that large proportions of the TSS consisted of ash. The ash content ranged from a minimum of 31 percent in December to 69 percent in April and February. On the average, 52 percent of the effluent TSS consisted of ash, an unusually high value relative to the 23 percent ash content of TSS in Woodland facultative pond effluents.

Volatile suspended solids--The effluent concentrations of VSS were substantially lower than the TSS, usually by a factor of 2. Only in December did the VSS exceed 20 mg/l.

Beginning with December, the relative proportions of algal and grazer biomass comprising the VSS were determined. Both in December and January about 85 percent of the VSS consisted of algal suspended solids (i.e. ash-free biomass passing through the straining fabric). This proportion declined to 45 percent in February. The balance of the VSS was made up of grazer biomass.

Chlorophyll a--Chlorophyll a was well correlated with algal SS (r^2 = 1.0). The minimum concentration observed, 8 µg/l, occurred in the February discharge while the highest, 281 µg/l, was observed in the December discharge. The average chlorophyll a was 96 µg/l (59 µg/l if December is excluded).

BOD_5--Despite its relatively high suspended solids content, the December effluent exerted a BOD_5 of only 10 mg/l. The average BOD_5 for December, January, and February was less than 10 mg/l. The highest BOD_5 measured in either isolation pond from offshore samples was 35 mg/l. This value was measured on a sample containing a VSS of 103 mg/l. Most samples taken offshore exerted less than 30 mg BOD_5 per liter.

COD--Total (unfiltered) COD ranged from 117 mg/l in December to 44 mg/l in May. Average total COD was 71 mg/l. The lowest filtered COD, 35 mg/l, was observed in February. Particulate COD--the difference between the total and filtered values--was well correlated with the VSS (r^2 = 0.9). Highest particulate COD, 49 mg/l, occurred in December and the lowest, 13-14 mg/l, in January and February.

Nitrogen--On the average about 70 percent of the nitrogen content of isolation pond effluents was organically bound. Ammonia and oxidized N made up the remainder in approximately equal proportions.

The total N concentration averaged slightly less than 5 mg/l. The December discharge contained the most fixed nitrogen, 8 mg/l. The remaining effluents had 3.7 to 5.4 mg N per liter.

Algal types--The three algal types most often found in the effluents were *Oscillatoria*, unidentified green algae, and *Chlamydomonas*. These algae were also prevalent in the inflows to the isolation pond (facultative effluents).

The greatest algal biovolume, 300 µm^3 x 10^6 per ml, was observed in December. This consisted mostly of *Oscillatoria*. In January the algal biovolume was also high, 130 µm^3 x 10^6 per ml, and again consisted mostly of *Oscillatoria*. The remaining effluents contained less than 40 µm^3 x 10^6 biovolume per liter with little or no *Oscillatoria*.

Influence of Wind--

The proportion of ash in the isolation pond effluents was greater than the approximately 10 percent ash usually found in algae grown under ideal conditions. It was noticed that samples taken from the isolation pond on windy days generally contained fine, greyish material which tended to clog the glass-fibre papers used in determining dry weights.

The ash and volatile components of suspended solids in daily effluent, composite samples were roughly correlated with daily wind movements. On days when the wind movement was 100 km per day or

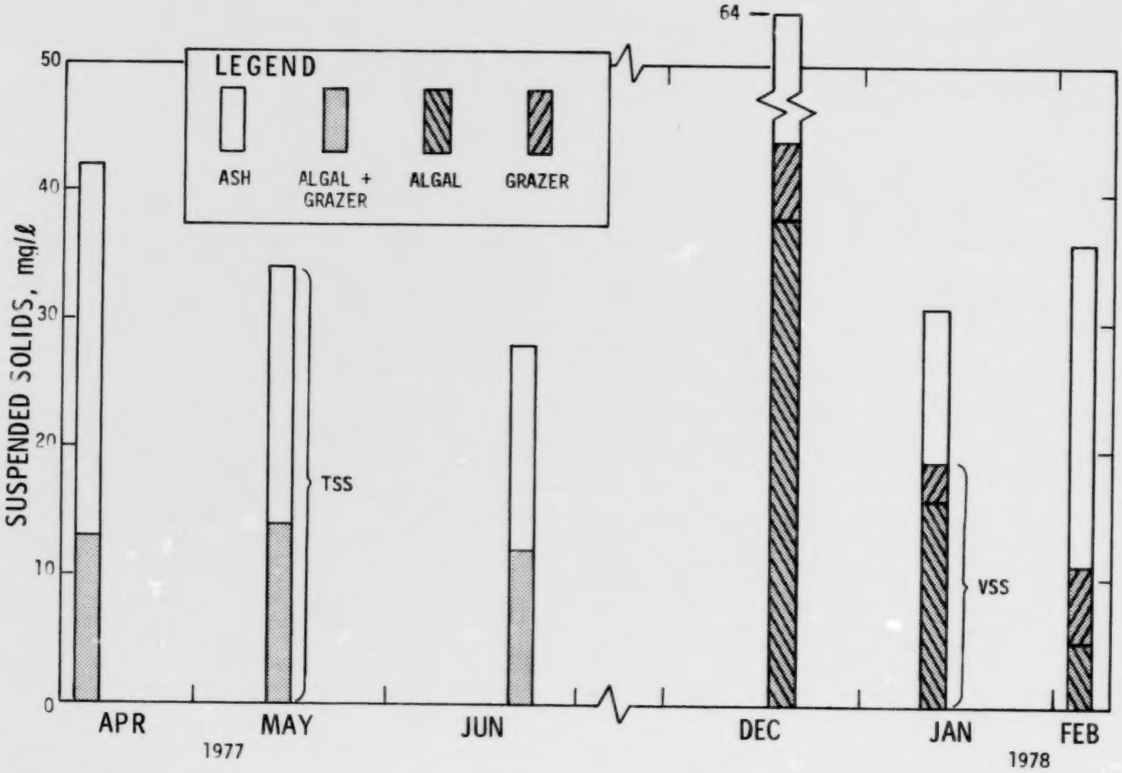

Figure 5. Total, volatile, algal, and grazer suspended solids in isolation pond effluents.

less, average TSS were 46 percent ash, whereas in samples taken on days of greater wind movement, ash made up an average of 60 percent of the TSS.

Isolation Pond Performance

A summary of the performance of the isolation pond in terms of removals of contaminants from facultative overflows is given in Table 7.

Isolation Cycle Duration--

The duration of each cycle was taken as the time between the start of a fill and the end of the ensuing discharge. As noted previously, 2 days were usually required to fill the isolation pond and 3 to 5 days were needed for its discharge. On the average, 22 days were allowed for each cycle.

Hydraulic Loading Rate--

Hydraulic loading rates were calculated by dividing the increase in pond depth at the beginning of a cycle (Δd) by the duration of the cycle. The average loading rate between 28 November and 5 February was 2.6 cm per day.

Suspended Solids Removal--

The percentage of TSS removed from facultative overflows was variable. During the two late spring cycles, the isolation pond was quite efficient, withdrawing almost 70 percent of influent TSS. However, in the January-February 1978 cycle, TSS in the isolation pond effluent were greater than in the influent. Overall 35 percent of influent TSS were removed in the isolation pond.

Chlorophyll a Removal--

Very high proportions of influent chlorophyll a were removed. As much as 95 percent of the initial chlorophyll a disappeared over the course of an isolation cycle. The overall removal was 80 percent.

BOD_5 Removal--

The average percent BOD_5 removed between 21 December and 5 February 1978 was 61 percent.

COD Removal--

Trends in the proportion of total COD removed paralleled removals of algal SS. The poorest removal occurred in the November-December cycle at the same time that the proportion of algal SS removed was at a minimum. The greatest removal was observed in April-May when the removal of algal biomass was also at its maximum. The overall percentage of total COD removed was 46 percent.

Total Nitrogen Removal--

The trends in nitrogen removal were similar to those in algal SS and total COD removal. The maximum value for nitrogen disappearance, 73 percent,

Table 7. Performance of the Woodland isolation pond.

Period	Duration Days	Hydraulic Loading Rate cm/day	Removals from Facultative Pond Overflows, %						
			Suspended Solids		Chlorophyll a	BOD_5	COD		Total Nitrogen
			Total	Algae			Total	Filtered	
April 18 - May 16 1977	28	--	69	85	95	--	77	--	73
June 6 - June 25 1977	19	--	67	82	83	--	59	--	42
Nov. 28 - Dec. 20 1977	22	3.3	9	33	54	--	13	--	19
Dec. 21 - Jan. 13 1978	23	1.8	45	65	78	50	52	34	37
Jan. 17 - Feb. 5 1978	19	2.7	<13>	71	92	72	31	10	18
Average	22	2.6	35	67	80	61	46	22	38

was achieved between April and May while the smallest, 19 percent, occurred from November to December. The average nitrogen removal was 38 percent.

Discussion

The overall performance of the Woodland facultative/isolation ponding system is presented in Table 8. On a seasonal basis, the facultative ponds were consistent in their ability to remove significant proportions of COD, BOD_5, and nitrogen from the influent sewage. The high rate of nitrogen subtraction, 70 percent, is particularly noteworthy as the removal of nitrogen in conventional sewage treatment plants can be accomplished only with additional equipment. The BOD_5 removal achieved in the facultative ponds, 87 percent, was marginally satisfactory with respect to EPA standards.

The level of treatment was considerably improved by the isolation pond. An overall BOD_5 removal of 96 percent was achieved between October and December. The extent of nitrogen removal was increased to 83 percent. The COD removal rate, which was 68 percent in the facultative ponds, was increased to 79 percent. Suspended solids of sewage origin were, of course, practically 100 percent removed. The proportion of ash in the isolation pond effluents was greater than normally observed in algal pond effluents. Higher ash contents were associated with high wind velocity as previously mentioned. This is shown in Figure 6 where the ash and volatile components of suspended solids in isolation pond effluents are compared to daily wind movements. On days when the wind movement was 100 km per day or less the average TSS was 46 percent ash whereas in samples taken on days of greater wind movement ash made up an average 60 percent of the TSS.

Table 8. Overall performance of Woodland ponding system.

Period	Removals From Raw Sewage, %					
	Facultative Ponds			Facultative & Isolation Ponds		
	COD	BOD_5	Total Nitrogen	COD	BOD_5	Total Nitrogen
April-June 1977	67	--	73	87	--	89
July-Sept. 1977	--	87	68	a	a	a
Oct.-Dec. 1977	69	88	70	71	96	78
Ave.	68	87	70	79	96	83

[a] No overflows from isolation pond during this period.

An explanation of the ash fraction is complex. A first assumption was resuspension of bottom sediments by wind since the higher ash contents were associated with high wind velocity. To check the bottom sediment resuspension hypothesis, a wave study was made. Observations of the isolation pond during windy days indicated that waves commonly reached 10 cm in height with lengths of about 50 cm and periods on the order of one second. When the ratio of basin depth to wave length is greater than one-half, waves behave as if they were in deep water. The isolation pond is about 150 cm deep when full, thus yielding a ratio h:L of 1.5. Horizontal velocity of water at the pond bottom, moving in response to waves of the size given above, was calculated by the method of Williams (14).

Substituting in the typical dimensions of Woodland's waves resulted in the Williams formula in a predicted velocity of 4×10^{-7} cm/sec. This result is much lower than the critical scour velocity of 3 cm/sec calculated from the equation of Camp (15) to be required to resuspend organic sediments. On the basis of these calculations, it appears that bottom sediment resuspension alone is not responsible for the higher ash content observed on windy days.

An alternative explanation which apparently fits the observed facts is that, due to localized high pH, marl may form in close association with or absorbed to the Oscillatoria cells and remain attached to the filament surface. Marl normally consists of a high fraction of calcium carbonate but also may contain aluminates and silicates. During windy periods, those Oscillatoria nearest the surface and therefore richer in ash become dispersed throughout the hypolimnion and a higher ash suspended solids would be obtained.

A third explanation is that during windy periods, bank erosion occurs and, due to circulation, fine clay becomes mixed into the hypolimnion. Studies are underway to determine if the ash contains a high percentage of calcium carbonate, but results thus far are inconclusive. There is, of course, the possibility that all of these phenomena

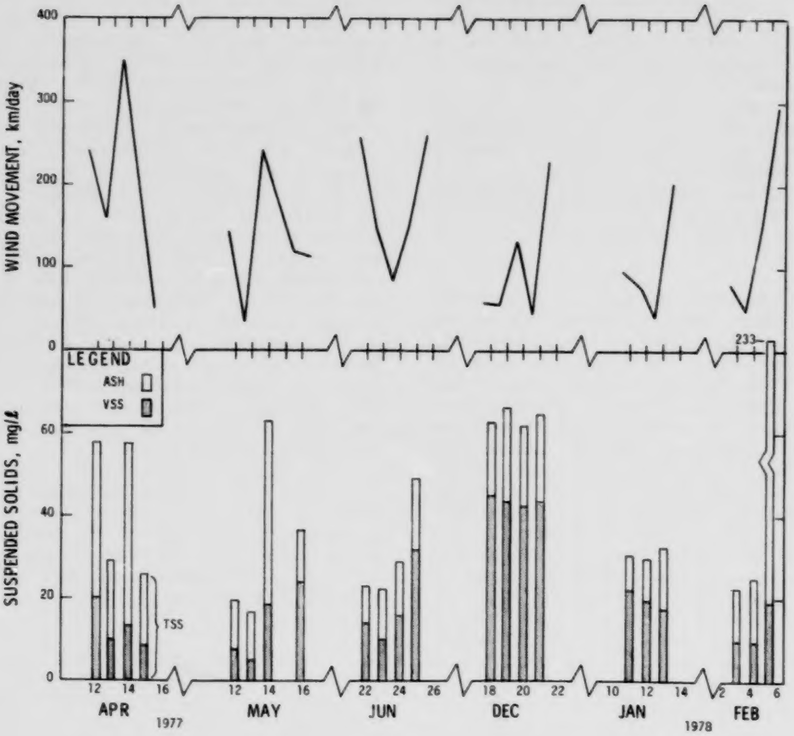

Figure 6. The influence of wind on the ash content of suspended solids in isolation pond effluents.

contribute to the observed high ash content.

The algal species important in the isolation ponds were for the most part the same species present in facultative effluents. In one notable exception, Closterium, a genus rarely observed in the facultative ponds, grew up in (isolation) Pond 5 to a concentration about twice that present initially. However, it disappeared within about a month.

The algal genus responsible for most of the 5-month long upset of the pond isolation process was Oscillatoria, which was also predominant in one of the facultative ponds. It is not understood at this time how these algae were able to grow to such high densities when ammonia-nitrogen concentrations had fallen below 0.5 mg/l. One possible explanation is that the release of nitrogen from pond sediments was sustaining the Oscillatoria. This possibility, however, conflicts with the observation that use of a previously empty pond for the isolation phase of treatment did not result in a decrease in algal concentration.

There is also the possibility that the Oscillatoria was able to anaerobically fix nitrogen from the atmosphere to satisfy its requirements. However, we were not successful in inducing N_2 fixation in samples of Oscillatoria taken from Woodland under conditions which did allow Plectonema, another non-heterocystis blue-green alga, to exhibit fixation.

One factor which does indicate the effectiveness of pond isolation in removing algae is the chlorophyll a concentration. Algae are normally about 1 percent chlorophyll a and reference to Table 6 indicates that the maximum chlorophyll a concentration--281 (micrograms per liter)--corresponds to an algal concentration of 28.1 mg/l.

INTEGRATED PONDING

Integrated ponding using the phase isolation concept involves optimum designs for the attainment of the most complete treatment possible in ponds. Ideally, the systems use continuous flow growth ponds in series, separation ponds which are either operated in series or parallel, continuously or in a batch. One example of an integrated ponding system presently functioning is in St. Helena, California. This system consists of a 2-acre facultative pond, a 5-acre high-rate pond, a 2-acre algal sedimentation pond, and two 5-acre maturation groundwater recharge ponds. We discuss here primarily those aspects which contribute to the algal separation.

Sedimentation, grit, and grease removal, together with methane fermentation and some biochemical oxidation, occur in the primary facultative pond of a series. This primary pond should be designed to permit thermal stratification to occur and to provide an anaerobic volume in which methane fermentation can occur with no intrusion of oxygenated surface waters. In such an environment, both methane fermentation and nitrate reduction occur. The primary pond processes of methane fermentation and nutrient reduction apparently greatly enhance the ultimate settleability of microalgae in the separation ponds because they contribute to diminishing the quantity of carbon and nitrogen available to the algae. Fine silt and other inert materials are also removed by sedimentation and autoflocculation in facultative ponds with the result that wastes entering the high-rate pond are freed of these materials and, hence, a greater fraction of the available light penetrates into the water and is available to the algae. As has been discussed before (4), photosynthetic oxygenation and biological oxidation occur most effectively in shallow mixed ponds, secondary to facultative ponds in the series. Ideally, in a shallow, mixed pond, algae grow up to their full light-limited potential, absorbing most of the remaining carbon dioxide and ammonia from the water, greatly enhancing their tendency to settle. An important influence of algal growth in the high-rate pond on separation is that they decrease the total alkalinity and diminish bicarbonate concentrations with respect to carbonate, therefore increasing the pH. This, in turn, tends to enhance formation of precipitates which aid in improving the settleability of the algae and, coincidentally, of the bacterial cells in the system. The higher pH levels and somewhat higher water temperatures induced by algae tend to decrease the solubility of waste components including calcium and magnesium salts, and also tend to precipitate heavy metals. Thus, some reduction in TDS and heavy metal content may be observed (particularly where evaporation is not high). The mixing characteristics of the high-rate ponding process improves the settleability of flocculent materials apparently by increasing floc size through particle entrapment and, perhaps, by other mechanisms. A second influence of algae in the high-rate pond is the removal of ammonium in direct proportion to their growth. Two factors are actually involved here--loss of ammonia to the atmosphere and uptake of ammonium by the algae. Ammonia loss to the air is enhanced by the mixing and is increased as the water temperature and pH increase. Algal growth brings about the high pH and, inasmuch as algae convert light to heat with high efficiency, also brings about a higher temperature in the water. The ammonium taken up by algae usually amounts to about 8 percent of their dry volatile solids.

The net result is that the high-rate pond either alone or in series with a preceding facultative pond or primary treatment system improves the settleability and, hence, the harvestability of microalgae in the system.

Detailed results of the separation of algae obtained in the high-rate ponding process have been published elsewhere and can be summarized as follows: In studies by Oswald and Romani (16), a facultative pond, followed by a high-rate pond and a covered sedimentation pond, gave more than 90 percent suspended solids removal in the settling pond. Work in the Philippines in 1977 (17), using screened raw sewage introduced to high-rate ponds followed by a settling pond, gave results indicating a minimum 75 percent suspended solids removal in the settling pond. In those studies, no facultative pond was used because the sewage available for the study was low in both BOD and nitrogen. On-going experiments at our laboratory with pilot high-rate ponds have demonstrated that over 70 percent of the algae could be removed by a continuous settling chamber operated at a detention time of only 12 hours. Our conclusion, therefore, is that with proper design and under favorable environmental conditions, either primary treatment or a facultative pond preceding a high-rate pond, followed by a short detention time settling pond will result in removal of 70 percent or more of the suspended algae in the high-rate pond effluent without chemical additives or special mechanical manipulation. The key is apparently the combination of carbon uptake, nitrogen loss, and mixing applied in the high-rate pond.

MICROSTRAINING FOR ALGAL REMOVAL

We have recently reported extensive work on the use of microstrainers for algal separation (18). Although stainless steel microstrainers used in the past for algal separation have been subject to fouling and corrosion (19), the recent development of non-fouling, non-corroding plastic screens shows considerable promise. Our current information is that such screens have a life expectancy of at least 2 years since several of our experimental units are of that age and still functional. Plastic screens, having apertures as small as 20 microns, are now available with throughputs of about 2 gallons per sq ft per minute. Throughput is, of course, proportional to aperture size and about 20 gallons per sq ft per minute will pass through a 50 micron rotary screen. The cost of microstraining at the 26 micron level has been estimated at about $50 (1977) per million gallons of liquid processed including the equipment interest, operations and maintenance, and all other cost factors. Microstraining costs decline somewhat with plant size, but the major item of cost is the straining surface itself which depends on throughput rates, mesh and particle size and concentration.

The key parameters in the effective operation of microscreens for algae removal from pond effluents is particle size distribution and concentrations. Particle sizes depend on the algal species that populate the ponds, their colonial aggregations and flocculation behavior. Obviously for microstraining to be effective, the algal cells, colonies or flocs must be significantly larger than the screen size. A relatively high concentration also helps algae removal as it results in formation of a "precoat" that helps to trap materials which are smaller than would normally be retained, allowing a greater variation in particle size distribution. High-rate pond systems allow both achievement of relatively high algal concentrations and some control over the algae population which could be used to selectively cultivate larger, filamentous or colonial. algae types readily removed by microstrainers.

Extensive experiments were carried out to test this concept using experimental, 12 m^2, high-rate ponds. The principal variables tested were detention times, specific biomass recycle, mixing speeds, and inoculation with desired algae types. The results demonstrated that it was feasible to maintain for considerable periods of time algal populations consisting of colonial Micractinium and Scenedesmus which could be efficiently (75 percent-95 percent) removed by microstrainers. However, after a few weeks to months upsets in the algal populations (e.g. change of species, grazing by zooplankton) led to a deterioration of effluent quality. Also, the pond operations that resulted in algal populations exhibiting good harvestability also resulted in poor algal biomass productivities. Thus, this approach to algal removal still requires further R & D and can presently be considered only in combination with other processes such as pond isolation or algal flocculation.

CONCLUSIONS

The investigation of pond isolation at Woodland suffered from a number of difficulties as is to be expected when a new process is tested on a large scale for the first time. Chief among these problems were lack of control over facultative pond loadings, wind-induced silt resuspension and absence of discharge during critical summer months. In spite of these problems it was shown that use of a draw-and-fill operated secondary (isolation) pond can allow significant removals of algae from facultative pond effluents. Possible upsets to the process by blue-green algae such as Oscillatoria need to be investigated further. It may be possible to limit the growth of these algae by increasing the loading to the facultative ponds during critical summer months.

Integrated ponding, where a shallow mixed, high-rate pond is used to produce settleable algal biomass which is subsequently removed in a deep, quiescent pond, requires a greater degree of operator attention than the facultative-isolation system. However, it becomes preferable in areas of higher population density because of its more efficient use of land.

In spite of high algae removal efficiencies, both systems appear to require a final polishing step to enable them to meet effluent standards more stringent than the general 30/30 EPA requirement. This step could take the form of natural gravel beds such as are used at Santee, California. Alternately, the process of intermittent sand filtration could be used. This method is particularly compatible with pond isolation because the effluent is drawn once every 20 days thus allowing a resting period for the filter. The entire filter could be flooded thus simplifying the distribution network required. Also, the length of filter runs could be extended because the bulk of organic matter (algae) would have been removed before reaching the filters. Some type of rapid sand filtration would be more suited for polishing effluents of integrated ponds, both to conserve land and because the flows are continuous.

The low operations and maintenance costs for ponds and the opportunity for energy conservation they allow relative to conventional, mechanical systems make these systems attractive wherever favorable climatic conditions exist. Present discharge standards, however, tend to discourage their application. In particular, pond systems and other aquatic plant based wastewater treatment systems should be subjected to discharge requirements which, without lowering overall pollution standards, take cognizance of the variations in performance standards that result due to seasonal and other climatic changes.

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AN INTEGRATED, CONTROLLED ENVIRONMENT AQUACULTURE LAGOON PROCESS FOR SECONDARY OR ADVANCED WASTEWATER TREATMENT

S. A. Serfling and C. Alsten*

INTRODUCTION

Conventional wastewater treatment lagoons have been limited in their application because of the high BOD and suspended solids which remain in the effluent in the form of algae, and their extensive land requirements. In an attempt to provide more process control and greatly reduce the land area requirements, engineers have developed a variety of relatively high technology treatment processes such as activated sludge, trickling filters, bio-disc, and elaborate advanced wastewater treatment processes. It is now becoming clear that these high technology solutions are not only failing in many cases to meet design or discharge standards but they are also very expensive to construct and operate, particularly in terms of energy consumption (EPA Report to Congress, 1975-76; Engineering News Record, 1978).

Both the conventional lagoon and the higher technology treatment processes have been designed for the purpose of sewage stabilization and disposal into the aquatic ecosystem. With increasing population densities, it is becoming obvious that what might appear to be a satisfactory disposal solution for one community becomes an environmental degradation problem for its neighbors. This problem, together with the increasing need for water and fertilizer, leads to the inescapable conclusion that wastewater must be dealt with as a resource to be managed rather than a problem to be disposed of. Taking this perspective then, wastewater treatment processes for the future must be designed to operate as managed ecosystems with controlled harvesting of nutrients, and planned reuse of the water either directly or indirectly by "downstream" ecosystems or communities.

It is, therefore, encouraging that the 1977 Clean Water Act requires that agricultural, aquacultural, and silvicultural alternatives for wastewater reuse be evaluated and encouraged as treatment mechanisms. The purpose of this paper is to present information on a managed ecosystem type lagoon system utilizing an integrated controlled environment aquaculture system for achieving secondary or advanced tertiary quality effluent.

Definition of Aquaculture Treatment Systems

Aquaculture, similar to agriculture, has typically referred to the farming of aquatic foods for direct or indirect human consumption (Bardach, Ryther and McLarney, 1972). By this definition then, wastewater would merely be a source of nutrient input, similar to animal manures, agricultural waste products or inorganic fertilizers. Although this is a common practice in developing countries (which often export the harvested fish or shellfish to the United States), this is not the type of aquaculture wastewater treatment addressed in this paper. Wastewater must first be "treated," and reuse of by-products is an optional activity of secondary concern. We propose as a preferred definition of aquaculture wastewater treatment (AQWT), a process in which aquatic species are intentionally stocked and harvested as part of a managed ecosystem for the purpose of removing wastewater nutrients and/or pollutants. The harvested by-products may or may not be used for any direct human benefit, such as organic compost, fertilizers, methane gas, methanol, or animal feeds (see Figure 1), but they are not intended for direct human consumption.

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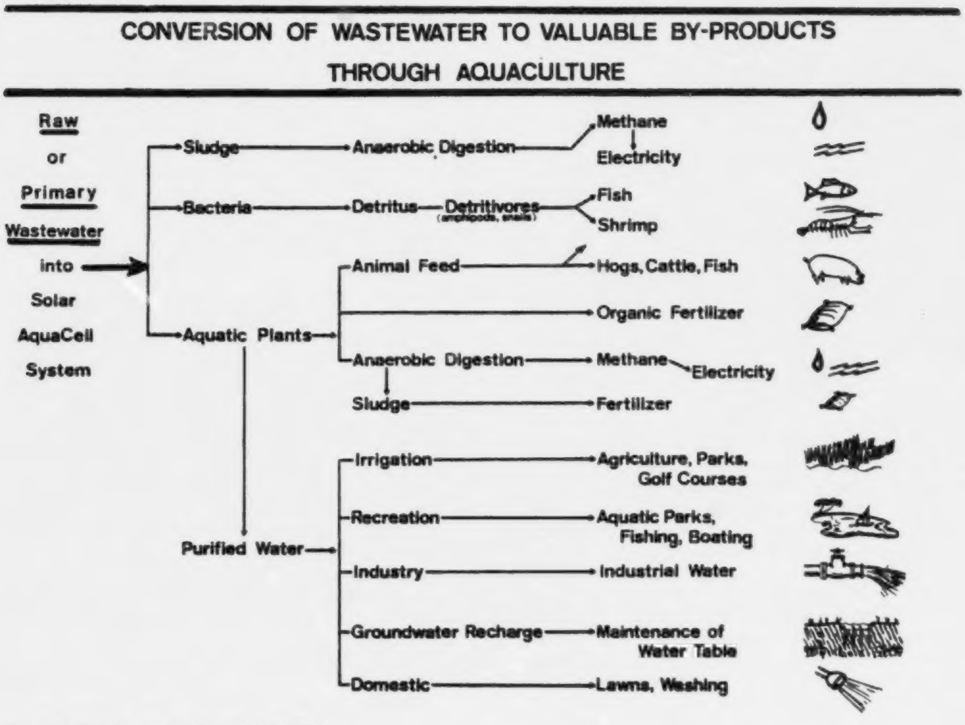
In a broad sense, all biological treatment processes could be considered aquaculture, since even the activated sludge process is entirely dependent on the growth, productivity, survival and harvesting of microorganisms for its treatment effectiveness. But due to the lack of intentional stocking or harvesting of organisms for an intended beneficial reuse, activated sludge does not fit the definition of aquaculture wastewater treatment. For similar reasons, algal lagoons which utilize mechanisms such as chemical precipitation to "harvest" the algae and return it to the pond for further degradation and stabilization, are not considered aquaculture. The distinction between the various means of aquaculture as it related to wastewater treatment has been well discussed by Duffer and Moyer (1978).

Limitations of Conventional Wastewater Treatment Lagoons

Wastewater stabilization or oxidation lagoons have proven successful in stabilizing wastewater by means of natural biological processes with low energy and maintenance requirements. In contrast to managed ecosystem processes, the objective of wastewater treatment lagoons has been the stabilization, rather than the permanent conversion and removal of the wastewater pollutants and nutrients. The design features of wastewater lagoons are thoroughly discussed by Oswald (1963) and Gloyna (1971). Unfortunately, the requirement for long retention times (typically 80-150 days) with large land areas, and the inability to achieve secondary or higher quality effluent on a reliable basis has restricted the application of wastewater stabilization lagoons.

Both problems, large land area requirements and poor effluent quality, are caused primarily by the dependence of the lagoon process on single cell algae. The biological concepts which describe the functioning of wastewater treatment lagoons has been thoroughly reviewed by McKinney (1974), in which he describes the cyclical and revers-


CONVERSION OF WASTEWATER TO VALUABLE BY-PRODUCTS
THROUGH AQUACULTURE
Raw
or
Primary
Wastewater
into
Solar
AquaCell
System
Sludge
Anaerobic Digestion
Methane
Electricity
Bacteria
Detritus
Detritivores
(amphipods, snails)
Fish
Shrimp
Aquatic Plants
Animal Feed
Hogs, Cattle, Fish
Organic Fertilizer
Anaerobic Digestion
Methane
Electricity
Sludge
Fertilizer
Purified Water
Irrigation
Agriculture, Parks, Golf Courses
Recreation
Aquatic Parks, Fishing, Boating
Industry
Industrial Water
Groundwater Recharge
Maintenance of Water Table
Domestic
Lawns, Washing

Solar AquaSystems, Inc. P.O. Box 88 Encinitas, CA. 92024

Figure 1. Potential wastewater conversion products and pathways using a managed aquacultural treatment process.

ible nature of the synthesis and endogenous respiration processes occurring with the algae, and aerobic and anaerobic bacteria communities within the pond. Under proper conditions algae can be highly productive, and rapidly assimilate and convert wastewater nutrients into a more publicly acceptable form, while producing oxygen in the process. However, the reverse cycle can just as easily and quickly take place. Under fluctuating conditions of solar input, temperature, nutrient loading, and concentrations of toxic substances, the algae die, decay and release the nutrients back to the water. Because of the inherent instability of algae, wastewater can be stabilized with an algal pond system only after long retention time (for example, see Middlebrooks et al., 1974). Recognizing the need for a managed ecosystem approach, McKinney (1974) concluded "the key to successful production of quality effluents from oxidation ponds lies in algae removal."

To take advantage of the rapid growth and oxygen producing features of algae, Oswald (1973) developed an accelerated algae culture system. This managed process utilized anaerobic primary treatment, shallow algae culture ponds to maximize sunlight penetration, long narrow channels to minimize short-circuiting, pre-inoculation of algae to quickly initiate the photosynthetic process, and various harvest mechanisms to remove the algae after only 2-4 days retention time. The controlled process reportedly works well, except for the difficulties encountered in attempting to reliably and inexpensively harvest the microscopic algae cells. Numerous other researchers have also encountered technical or economic limitations with micro-screening, chemical coagulation, centrifugation, sand filtration, or pond isolation techniques for removing algae (for example, see Benemann et al., 1977).

Because of the widely fluctuating performance of algae based ecosystems, particularly under conditions of seasonal changes and climatic differences in different geographic zones, engineers have experienced difficulty in developing or utilizing reliable mathematical models for the design of algae lagoon systems. Other engineers have greatly reduced the land requirement for wastewater treatment by utilizing expensive, high technology processes such as activated sludge, trickling filters, and bio-disc processes to obtain better control under high loading rate, short retention time conditions. Yet, process instability and poor effluent quality still remain a problem (EPA Report to Congress, 1975-76, and Engineering News Record, 1978). Once again, the problem is due primarily to the instability of unicellular type organisms, (in this case, bacteria and protozoa) as the sole biological component for treatment.

Aerated lagoons have been developed to provide a good compromise between the low technology stabilization lagoons and the higher technology activated sludge, trickling filter or bio-disc processes. The assurance of constant oxygen levels allows much higher loading rates without process breakdown or odor problems, but the effluent quality from most aerated lagoons still fails to meet EPA discharge criteria because of the high quantity of algae cells remaining in the effluent.

UPGRADING LAGOONS USING FLOATING AQUATIC MACROPHYTES

Advantages of Floating Macrophytes

Floating aquatic macrophytic plants such as water hyacinths (Eichhornia crassipes) or duckweeds (Lemna spp., or Spirodela spp.) offer the following advantages over suspended algae for treating wastewater:

1) Stability and Hardiness: Extensive studies by Wolverton (1978) with water hyacinths, and Hillman and Culley (1978) and Harvey and Fox (1973) with duckweeds have demonstrated that both plants are very stable and hardy organisms that can survive and rapidly multiply under varying environmental conditions. In fact, the plants have been shown to greatly increase both average and total productivity when cultured in wastewater treatment ponds. They can tolerate widely fluctuating nutrient loading rates, solar input, water and air temperature changes, fluctuations in water chemistry such as pH, carbon dioxide, alkalinity, and are not affected by toxic compounds (heavy metals, chlorinated hydrocarbons, etc.) at concentrations typical in most municipal sewages. Studies by Wolverton (1978) have shown that water hyacinths can be used for concentration, removal, and potential recycling of a variety of heavy metals occurring in many industrial effluents. A good review on the use of aquatic plants for wastewater treatment is provided by Duffer and Moyer (1978).

2) Provide Shade to Prevent Algae Growth: Floating aquatic macrophytes eliminate high effluent levels of suspended solids and BOD caused by algae cells because the sunlight penetrating through the plant fronds is insufficient in most situations to allow significant phytoplankton growth. For this reason,

aquatic macrophytes are showing good potential as "polishing" ponds to be added on to conventional treatment lagoons or higher technology processes (Cornwell et al., 1977).

3) Rapid Growth, High Productivity: Water hyacinths and duckweeds have been shown by numerous researchers to be extremely fast-growing and, thus, have the capability for removing large quantities of sewage nutrients in a relatively short time and small land area. It is a relatively simple concept that in order to clean wastewater, one must remove from it the chemicals originally put into the water during its use, and the quantity removed must correspond more or less to the quality of effluent desired. Thus, the productivity of the cultured biomass bears a direct relation to the ability of the system to purify water.

4) Ease of Harvest: The large size of the floating aquatic macrophytes makes their removal relatively easy in comparison to unicellular algae. Water hyacinths do present a difficult harvest situation if they have been allowed to grow uncontrolled for many months, resulting in heavily intertwined plants which resist removal. Harvesting on a regular schedule reduces this problem significantly. Duckweeds are much easier to harvest because of their smaller size and absence of entangling roots or stolons. They can be removed by simple surface skimming devices as part of the pond overflow mechanism.

5) High Reuse Potential: Water hyacinths and duckweeds offer numerous reuse possibilities, including high protein animal feeds, conversion to rich organic compost or fertilizer, conversion to methane gas, or methanol. Water hyacinths grown in sewage lagoons typically average 20 percent protein (Wolverton et al., 1978), while duckweeds have been shown to average over 40 percent protein (Hillman and Culley, 1978). Studies by Wolverton (1978) and Lecuyer and Marten (1975) have demonstrated that one acre of water hyacinths can produce an average of 200,000 to 300,000 cubic feet of methane gas per year. Tables 1 and 2 summarize the yields and quantities of aquatic plants

Table 1. Comparison of solids production for aquatic macrophyte vs. conventional systems.

		Ave. Quantity Per Million Gallons			
		Wet Weight (lbs)	Dry Weight (lbs)	Cubic Ft.	% Moisture
1.	Imhoff Tank (Primary)		690	67	85
2.	Primary Sedimentation (Undigested)	25,000	1,250	390	95
3.	Activated Sludge (Secondary, undigested)	150,000	2,250	2,580	98.5
4.	Activated Sludge & Primary (Anaerobically digested)	7,000	1,400	360	94
5.	Chemical Precipitation (De-watered-vacuum filter)	12,000	3,300	193	72.5
6.	Water Hyacinths (AquaCell lagoon, 1 acre/mgd @ 40 dry ton/acre/year)	4,400	220	160 (chopped) 100 (compost)	95
7.	Duckweeds (AquaCell lagoon, 2 acres/mgd)	1,600	80	26	95
8.	Algae Lagoon (50-100 day retention time)	416-3,333	20-166	7-52	95
9.	High Rate Algal Lagoon (2-3 days retention time)	4,400-6,000	220-300	68-93	95

Note: Data on
1-5. From W.P.C.F. (1977).
6. After Wolverton et al. (1975) and Lecuyer et al. (1975).
7. Myers (1977).
8. Metcalf and Eddy, Inc. (1972) and Benemann et al. (1977)
9. Benemann et al. (1977).

and their conversion products that are obtainable under various wastewater treatment conditions.

6) Advanced Tertiary Quality Effluent Can Be Economically Achieved: Due to the stability, high productivity, shading effect, and relatively easy harvest, floating aquatic macrophytes can produce advanced tertiary quality water with little capital or operating costs. The plants convert dissolved nutrients into a fixed biomass which is stable until harvested, and does not recycle or remain in the effluent or pond bottom to create problems at a later time. This controlled aquaculture process does not require the expensive energy, chemicals, construction or maintenance cost that is typically utilized as a form of "brute force" treatment by conventional high technology advanced wastewater treatment systems.

Disadvantages of Aquatic Macrophytes

In spite of the numerous advantages of aquatic macrophytes, several disadvantages exist which have limited their usefulness to date. Water hyacinths are extremely hardy under warm, humid conditions, but because of their tropical nature, cannot survive cooler winter temperatures during which wastewater treatment must still be achieved.

Table 2. Production quantities of aquatic plants and their conversion products.

	Quantity of Harvest Per Acre (Wet)[h]				Converted to Compost (80% H_2O Loss)		Converted to Methane Gas
		Volume					
	Weight lbs/ac/day	Whole ft^3/ac/day	Chopped ft^3/ac/day	Dry Weight lbs/ac/day	Volume yd^3/ac/day	Weight lbs/ac/day	Volume[d] ft^3/ac/day
1) Water Hyacinth[a]							
- Summer	10,000	1,000	250	500	6	2,500	1,750
- Winter	1,320	131	32	66	0.7	320	230
Average Daily Production	4,400	440	110	220	2.5	840	770
Density[b]		5-15 lbs/ft^3	30-50 lbs/ft^3		15 lbs/ft^3		
% Moisture	95%	95%	95%	-0-	67%	-0-	-0-
2) Duckweeds[c]							
- Summer	1,095	18		54	0.48	273	325[f]
- Winter	550	9		27	0.25	135	162
Average Daily Production	850	14		42	0.37	197	252
Density		60 lbs/ft^3 [e]			20 lbs/ft^3 [g]		

Notes:

[a] Based on a composite of data from Wolverton (1978); Ryther et al. (1978); and A.A.R.I. (1978).

[b] After Bagnall (1975).

[c] Based on a mixed culture of duckweeds, after Myers (1977).

[d] After Wolverton et al. (1975); Lecuyer et al. (1975).

[e] Emperical measurement (A.A.R.I.).

[f] Estimated 6 ft^3/lb dry duckweed.

[g] Estimated.

[h] These production figures are for water hyacinths alone or duckweek alone and are not additive in combined culture.

Even in southern climates such as Mississippi and Texas, water hyacinths die back during the winter and lagoon treatment efficiency suffers (Wolverton, 1978; Dinges, 1978). In many locations water hyacinths are considered a noxious weed, and any environmental concerns must be evaluated before their use.

In contrast, duckweeds are well known for their ability to survive and grow in cold climates, and there are numerous species known throughout the world in all climate zones. Duckweeds are limited by two main problems: they are highly prized as food by most fish and waterfowl, and can be quickly consumed in any open pond; and they are easily blown by winds upon the pond shore. Consequently, duckweeds are not reliable as a treatment method unless provision is made to eliminate predators and construct long narrow ponds with high banks or wind screens to prevent surface disruption, as suggested by Hillman and Culley (1978).

Evaporation losses from macrophytic plants can also be a serious problem in areas with a shortage of water. Del Fosse (1977) in reviewing literature on water hyacinth culture, states that researchers have shown water hyacinths increase evapotranspiration water losses by as much as 2 to 7 times normal rates, or the equivalent of 3 to 12 acre feet every six months. Because of the problems encountered with low winter temperatures, predators, wind effects, and evapotranspiration, all of the chief researchers on floating macrophyte plants have stated that greenhouse pond covers would be desirable and perhaps essential in order to allow macrophyte systems to be more adaptable to different environmental conditions (Wolverton et al., 1978; Dinges, 1978).

ADVANTAGES OF THE SOLAR AQUACELL CONTROLLED ENVIRONMENT PROCESS

The integrated, controlled environment aquaculture treatment system described in this report is the outgrowth of a five-year research development program which has concentrated on the integration of five different but relatively well-proven technologies. These technologies have been incorporated into what is known as the Solar AquaCell Process in order to overcome the abovementioned problems and combine the best features of low construction and operating costs of aerated lagoons, with the process control, advanced treatment capability, and reduced land requirements of conventional high technology treatment processes. The intention was to develop a more reliable, shorter retention time lagoon process that could extend the applicability of lagoon systems to communities otherwise forced to construct expensive and energy intensive, high technology systems.

The five main technologies integrated into the system (Figures 2 and 3) are: 1) The basic multicell lagoon process; 2) the use of floating aquatic macrophytes, particularly water hyacinths and duckweeds in combination; 3) greenhouse covered ponds to provide insulation and transfer of solar heat to the water; 4) high surface area fixed-film substrates (Activated Bio-Web Substrates) to provide habitat for the bio-film and associated microorganisms and invertebrate detritivore community; 5) a dual aeration and solar heat exchange system for maintaining proper dissolved oxygen concentrations, transferring solar heat from the air phase to the water phase in order to increase metabolic rates and treatment efficiencies, and providing for a gentle stirring and partial mixing of the wastewater past the aquatic plant surface area and the bio-film substrates.

Historical Development of the System

Development of the Solar AquaCell process began in 1972 using the five basic elements of the system for treating and recycling wastewater from intensive aquaculture food production systems designed for producing freshwater fish and shrimp. Commercial fish feeds, and cattle and chicken manures were used as the nutrient input to tanks containing high surface area plastic substrates, floating aquatic macrophytes, diffused aeration, and greenhouse covers. A variety of microorganisms, protozoa and invertebrate detritivores (amphipods, ostracods and snails) were added to the system to recycle particulate wastes back to the fish and shrimp. The organic input and waste production load added to the system each day was qualitatively and quantitatively very similar to domestic sewage except for phosphates, which are much higher in sewage due to the use of household detergents.

Studies conducted with these systems over a three-year period demonstrated that the integrated process was highly stable, and that essentially all of the waste products and nutrient input could be completely recycled through food chain processes to be harvested in the form of fish, shrimp, and aquatic macrophytes. It was a particularly interesting observation that the bacteria/detritus/detritivore food chain pathway was considerably more efficient at converting and recycling nutrients than was the photosynthetic plant pathway. Studies of nitrogen flow within the system indicated approximately 75 percent was recycled through the bio-film/detritus/detritivore pathway under optimal harvest conditions by the floating aquatic macrophytes (Serfling, 1976). Under these intensive culture conditions, BOD and suspended solids were typically maintained at levels below 20 parts per million, while ammonia and nitrite were less than 0.1, and total nitrogen less than 15 parts per million. The greenhouse tank covers functioned well during the colder winter periods and allowed continued growth of the aquatic macrophytes and other food chain organ-

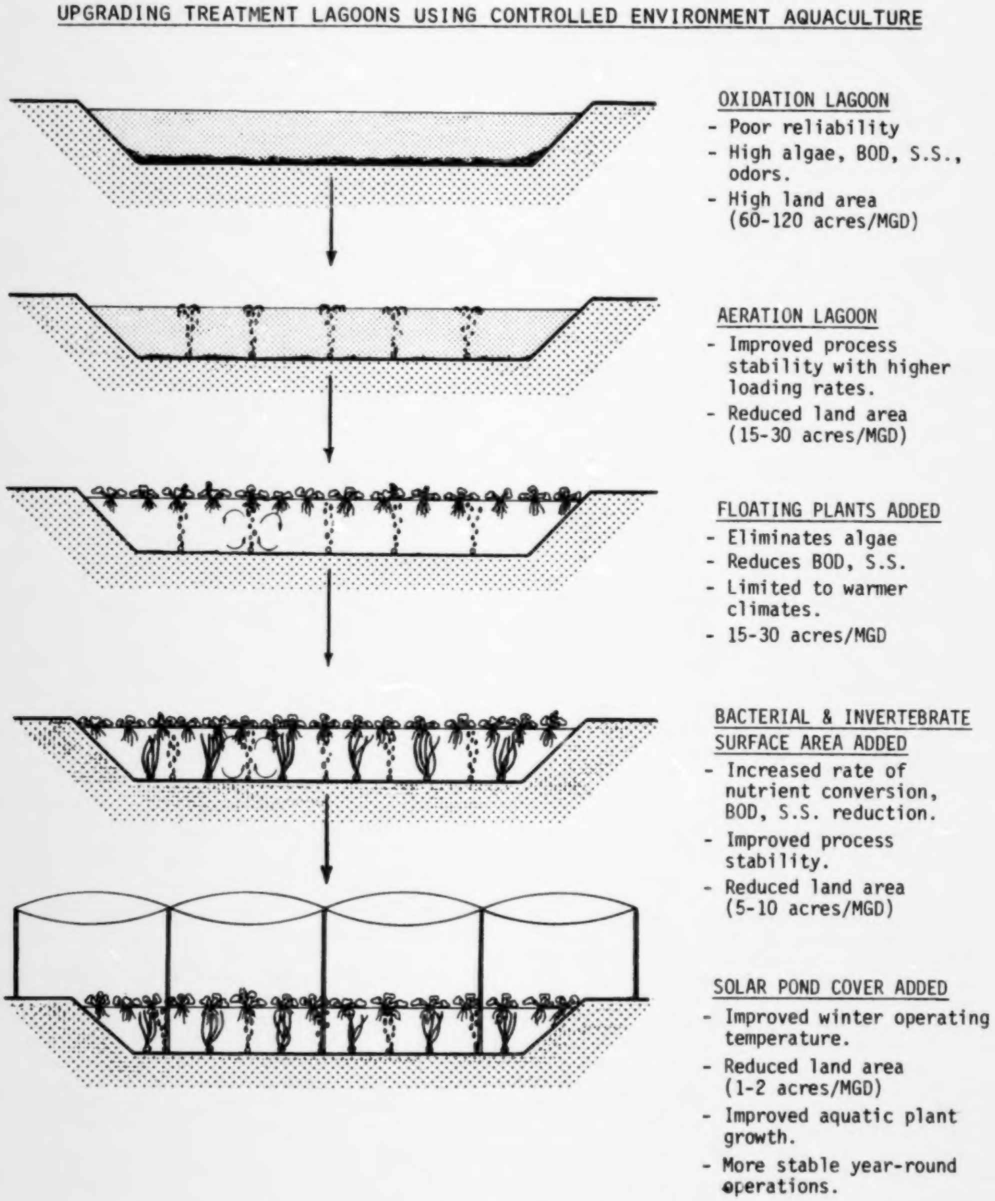

UPGRADING TREATMENT LAGOONS USING CONTROLLED ENVIRONMENT AQUACULTURE
OXIDATION LAGOON
- Poor reliability
- High algae, BOD, S.S., odors.
- High land area (60-120 acres/MGD)
AERATION LAGOON
- Improved process stability with higher loading rates.
- Reduced land area (15-30 acres/MGD)
FLOATING PLANTS ADDED
- Eliminates algae
- Reduces BOD, S.S.
- Limited to warmer climates.
- 15-30 acres/MGD
BACTERIAL & INVERTEBRATE SURFACE AREA ADDED
- Increased rate of nutrient conversion, BOD, S.S. reduction.
- Improved process stability.
- Reduced land area (5-10 acres/MGD)
SOLAR POND COVER ADDED
- Improved winter operating temperature.
- Reduced land area (1-2 acres/MGD)
- Improved aquatic plant growth.
- More stable year-round operations.

Figure 2. Upgrading wastewater treatment lagoons by use of controlled environment aquaculture processes.

isms throughout the winter. Solar heated tank temperatures in the San Diego climate averaged 75-85°F, whereas outside uncovered pond water temperatures averaged 40°F during the same winter months.

Designing the Controlled Environment Aquaculture Process for Sewage Treatment

Based on the success experienced with the high loading rates utilized in the controlled environment aquaculture system incorporating the integrated biological and physical components, the system was modified to test its performance using domestic sewage as the nutrient input source. The major objective in modifying the system for wastewater treatment was to incorporate biological and physical components in a manner which would maximize process stability and reliability on a year-round basis. This objective was considered most important because it is the main shortcoming of both conventional lagoon processes and conventional high technology processes to date. A second major design objective was to combine the best design and

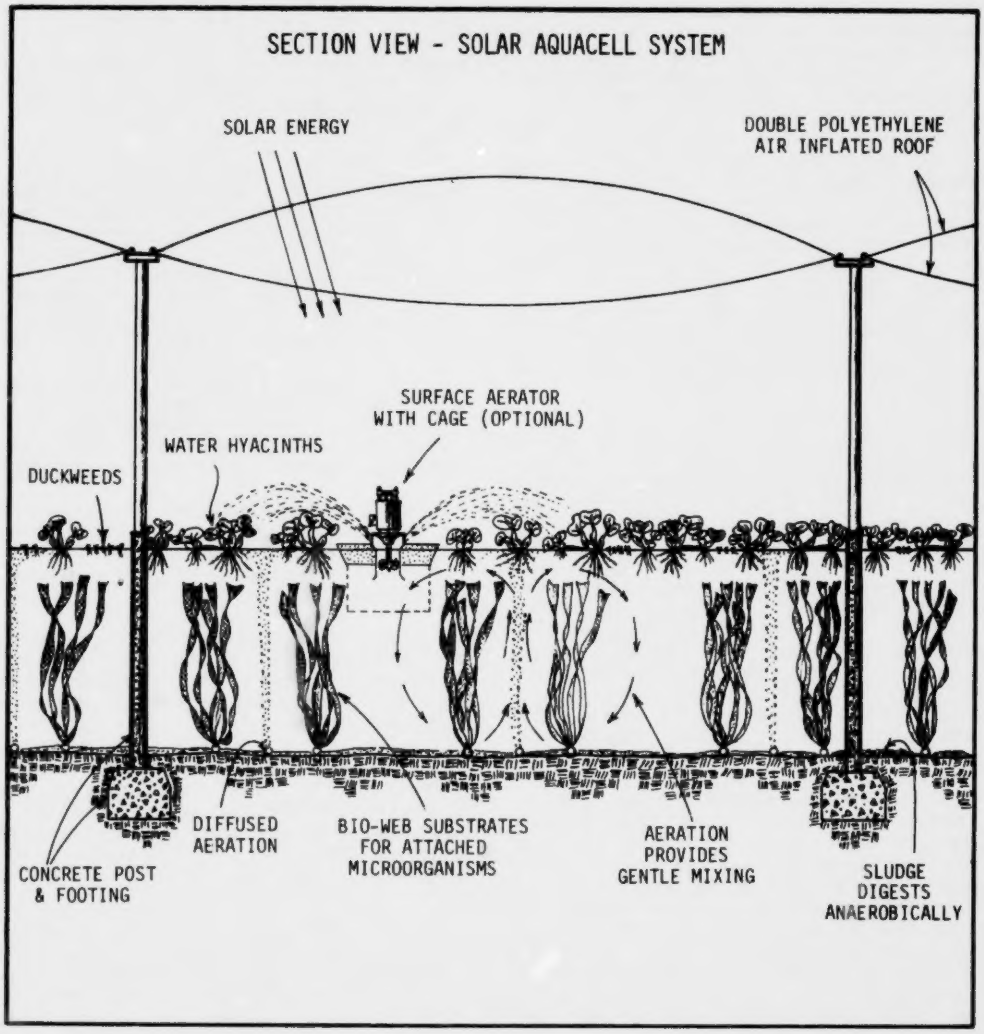

SECTION VIEW - SOLAR AQUACELL SYSTEM
SOLAR ENERGY
DOUBLE POLYETHYLENE AIR INFLATED ROOF
SURFACE AERATOR WITH CAGE (OPTIONAL)
WATER HYACINTHS
DUCKWEEDS
DIFFUSED AERATION
BIO-WEB SUBSTRATES FOR ATTACHED MICROORGANISMS
AERATION PROVIDES GENTLE MIXING
CONCRETE POST & FOOTING
SLUDGE DIGESTS ANAEROBICALLY

Figure 3. Section view of solar AquaCell lagoon system.

operating features of the "soft" technology lagoon processes with their low construction and operating cost, with the reduced land requirement of the more controlled, but expensive "hard" technology systems. The hybrid process could then encourage the use of natural biological lagoon systems for communities unable to afford, or without access to the large land areas required for conventional oxidation ponds, and thus forced to install high technology systems. The function and design objective of the various components of the system are discussed in more detail as follows.

Bio-Film Substrates

It is common practice in aquaculture systems as well as wastewater engineering to utilize high surface area substrates for the attachment of nitrifying bacteria, and waste-consuming microorganisms and invertebrates. In trickling filter, bio-disc, or activated sludge processes, simple materials such as rocks, redwood lathe boards, fiberglass plastic sheets, or sewage sludge itself is proven technology for providing substrate for attachment of bio-film microorganisms. Surface area requirements for conventional trickling filter or bio-disc processes typically range from 0.3 to 1.0 sq. ft. of total available surface area per gallon of wastewater treated per day, while retention times are only 4-6 hours to achieve secondary treatment (WPCF, 1977).

Based on our experience with acquaculture food production system, we have found that plastic mesh webbing material anchored on the bottom and allowed to extend upwards through the water column provides a simple, inexpensive, yet very effective and durable substrate for bio-film and invertebrate growth (Figure 4). The function of the Bio-Web substrate is similar to the other well-proven conventional processes. However, because of the relatively low cost of earthen ponds and the freely suspended Bio-Web substrates. it is economical to design lagoon processes with much higher bio-film surface area to loading rate ratios and with longer retention times. From our studies, we have found that design factors of 2-4 sq. ft. of surface area/gallon treated/day, and 2-4 days retention time are adequate for achieving secondary to advanced tertiary treatment (see following sections). Thus, the combination of lagoons with bio-film substrates can result in a design that is more conservative, reliable, and economical than high technology systems for producing either secondary or higher quality water.

Adding Bio-Web substrates to conventional lagoons, which have only the pond bottom as surface area for attached microorganisms, can increase this attachment area from 20 to 60 times. In terms of bio-film treatment processes only, the retention time could be reduced to one-twentieth to one-sixtieth of that required by conventional lagoons (this estimate has been fairly well supported by our experimental results, as discussed later).

From a design viewpoint, the advantages of using biological substrates in lagoon systems can be summarized as follows, and as shown in Figure 4.

1) Reduction of BOD and suspended solids by physical means, with the Bio-Web substrates acting as a barrier to cause coagulation and sedimentation of the suspended organics, which are then digested anaerobically on the pond bottom.

2) The vertically suspended, buoyant Bio-Web substrates maintain the bacterial film in suspension without the need for electricity (e.g. for aeration), as is required for the complete mix activated sludge process. Similarly, the trickling filter process requires high pumping costs to lift and recycle sewage through the trickling filter media.

3) Elimination of ammonia from the effluent, which otherwise exerts a high biological oxygen demand on receiving waters; and which can combine with chlorine and produce toxic chloramines.

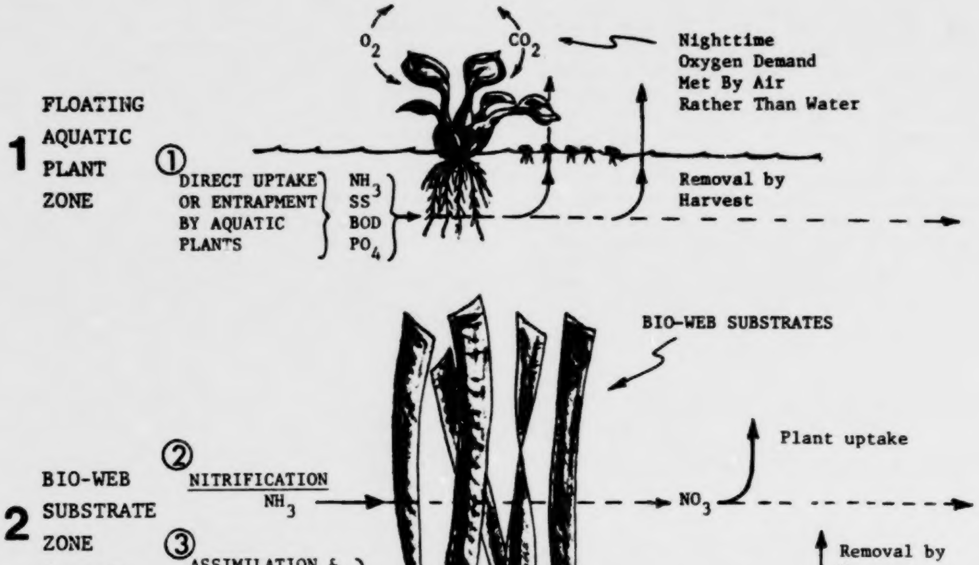
4) Nitrification of toxic ammonia to non-toxic nitrate to allow growth of organisms in succeeding cells, or if reuse of nutrients of aquaculture food production is desired.

5) By varying retention time, the Bio-Web substrates can aid in producing either secondary or advanced tertiary quality effluent from the same lagoon system.

6) The Bio-Web substrates also provide extensive habitat for grazing invertebrate detritivore organisms which consume, concentrate, and metabolize the organic and inorganic detritus material adhering on the Bio-Web substates.

7) No structures are required to support the Bio-Web, since it is a low-density, floating material and only needs to be anchored at the bottom. It can be easily added to a pond system without the cost of additional components.

Because our initial studies had indicated that the great majority of nutrients and organic matter was recycled through the bacteria/detritus/detritivore food chain rather than the aquatic plant food chain, it was clear that the Bio-Web substrate were the most important elements in the integrated AquaCell system, and their optimization would need to be carefully considered.



O_2
CO_2
Nighttime Oxygen Demand Met By Air Rather Than Water
1 FLOATING AQUATIC PLANT ZONE
①
DIRECT UPTAKE OR ENTRAPMENT BY AQUATIC PLANTS
NH_3
SS
BOD
PO_4
Removal by Harvest
BIO-WEB SUBSTRATES
2 BIO-WEB SUBSTRATE ZONE (AEROBIC, PARTIAL MIX)
②
NITRIFICATION
NH_3
NO_3
Plant uptake
③
ASSIMILATION & METABOLISM OF ORGANIC NITROGEN & CARBON
Detritivores
Removal by Harvest With Plants
Detritus (Bacteria & Fecal Matter)
④
COAGULATION, PRECIPITATION & SEDIMENTATION OF ORGANIC AND INORGANIC SUSPENDED SOLIDS (NO O_2 REQUIRED)
Detritus To Anaerobic Sediments
3 ANAEROBIC ZONE
⑤
HIGH SEDIMENTATION IN QUIESCENT ZONE
N_2, CH_4, CO_2
⑥
ANAEROBIC SLUDGE DIGESTION

Figure 4. Conceptual section view of the AquaCell pond showing three treatment zones and function of Bio-Web substrates. BOD and nitrification aeration requirements are reduced by harvesting, sedimentation, and anaerobic digestion processes.

Solar Greenhouse Pond Cover

Because wastewater treatment processes must operate under all weather conditions every day of the year, one must design an aquaculture system that is not susceptible to upset by freezing conditions, invasion by predators which might consume the aquatic plants, or windstorms destroying the surface plant cover by blowing it onto the shore. Even if occurring only once a year, these events would upset treatment operations until regrowth occurred. This could take several months during winter periods.

Independent of the above risk considerations, the cost savings provided by a solar pond cover, in terms of reduced costs for land and pond construction, can easily justify its use (Table 3). By maintaining higher pond temperatures during the winter periods, metabolic rates of bacteria, microorganisms, detritivores, invertebrates and aquatic plants will be signifcantly increased, which in turn greatly reduces the retention time and land area required. For example, it is well recognized that metabolic rates of most organisms usually increase by a factor of 2.0 for every 10°C rise in temperature (Barnes, 1937). This varies of course, depending on the species and its physiological tolerance ranges, but on the average can mean that a pond maintained at 15°C (59°F) will metabolize twice the sewage load as an uncovered pond operating at 5°C (40°F).

Since most domestic sewage enters the treatment facility at relatively warm temperatures during winter periods due to the extensive use of hot water in most homes, the greenhouse pond cover need only act as an insulation barrier to maintain relatively warm pond operating temperatures. Combined with the solar

Table 3. Comparison of average construction costs for the solar AquaCell versus other lagoon systems for secondary treatment.

	Costs Per 1.0 mgd Facility ($1,000)		
	Oxidation Lagoon	Aerated or Water Hyacinth Lagoon	Solar AquaCell Lagoon
Retention Time	60 - 120 days	12 - 25 days	2.0 - 3.0 days
Land Area Requirements (water surface)	50 - 90 acres	8 - 20 acres	1 - 1.5 acres
Land Costs/Acre[b]	$2 - 5	$2 - 10	$5 - 10
Pond Construction $/Acre (includes piping, sealing, gravel rock, access, etc.)	$3 - 5	$8 - 15	$25
Total Land & Pond Construction Cost	$225 - 900	$80 - 500	$30 - 40
Other Components			
- Aeration System and/or Harvesting Equipment (screening or sand filtration may be necessary)	-0-	$40 - 150	@ $350,000 acre average cost = $350 - 525
- Greenhouse, Bio-Web Substrates, Anaerobic Pond Covers & Gas Collection, Solar Heat Exchange Systems, etc.	-0-	-0-	
TOTAL LAGOON SYSTEM COST[a]	$225 - 900	$120 - 650	$350 - 565

[a] Costs are for the lagoon system only, and do not include other variable costs such as engineering, administration, headworks, sand filtration or screening, disinfection, pump station, etc. Assumes a minimum 3-Cell system for each process.

[b] Land costs are highly variable. It is assumed oxidation lagoons would be most suitable at lower land costs, while the AquaCell system would be better in higher land cost situations.

heat transfer, which can be considerable even on cloudy days, it is possible for a greenhouse covered pond system in northern climates to have average operating temperatures of 12-17°C (54-63°F) during a 4 day retention period. In contrast, uncovered ponds have operating temperatures of only 2-4°C (36-40°F). Based on the metabolic rates of the organisms, it is thus theoretically possible to reduce the retention time and land area to one half to one fourth that of the exposed pond, due to improved operating temperatures alone.

This reduction in retention time can be carried further. During winter periods when conventional lagoons are at near-freezing temperatures, biological activity and algal growth are essentially stopped, yet the effluent quality in terms of BOD and suspended solids is satisfactory, primarily due to the lack of algae. However, the accumulated nutrients stored during the winter period must then be digested during the spring period of rising temperatures, along with the continuously incoming sewage. The greatly increased loads during spring usually cause tremendous algae blooms, and subsequent massive algae die-offs, which create high oxygen demand and anaerobic odor problems. Because of these cyclical winter and spring upset problems, conventional lagoons must be designed 2-3 times larger than otherwise required. The greenhouse pond cover can greatly alleviate this problem by maintaining higher winter operating temperatures to allow more consistent biological treatment of the sewage, and thus avoid the large accumulation of sludge and biological oxygen demand.

The high evapotranspiration rates of aquatic macrophytes can increase water losses of uncovered ponds from 2-7 times the normal rate, or about 0.5 to 2.0 acre feet per month (Del Fosse, 1977). In water-short areas this can be a considerable financial loss, and in areas with high dissolved salts in the wastewater, the increased salt content after evaporation can render the water useless for irrigation. With the use of a well-sealed greenhouse pond cover, the evaporative losses are reduced to insignificant amounts, and the total dissolved salts can be maintained or even reduced, due to the net uptake of minerals by the aquatic plants and pond invertebrates. Thus, the greenhouse pond cover can be very cost effective under conditions where water is a valuable commodity.

In cold-climate situations, snowloads are not a serious concern because the warm temperatures within the greenhouse (including between the two plastic roof membranes), and the pond, which acts as a giant heat reservoir, quickly melt any snow accumulating on the roof membrane. Greenhouses are commonly used in the hydrophonic vegetable and decorative plant industry in northern climates without experiencing serious snowload problems. Greenhouse systems utilizing douple-layer, air-inflated polyethylene roofs have particular advantages in retaining pond heat. It has been demonstrated that such roofs can reduce heat losses to 50 percent of greenhouses with single-layer roof glazings consisting of glass, fiberglass, or polyethylene (Keveren, 1973).

RESULTS OF THE SOLANA BEACH PILOT LABORATORY

Description of the System Design and Operation

A pilot AquaCell system was constructed in Solana Beach, Calif., in August 1976 to demonstrate process performance using domestic sewage. The system consisted of a series of four tanks averaging 1500 gallons each, and 6' x 8' x 4.3' deep in size. Each tank was constructed using wood framing, a hypalon rubber liner, and plumbed for independent sampling of each stage. A polyethylene cover over the tank retarded evaporative and heat losses, and the entire treatment facility was contained within a 25' x 50' greenhouse.

The source of sewage was a small sewer line transversing the property where the facility was constructed. Sewage was pumped into the facility using a submersible pump and flows were adjusted to average 1500 gallons per day to achieve approximately one day retention time per cell, or four days total. Because the sewer line serviced only about 18 houses, it often ran dry during the late evening and early morning hours, causing considerable problems in pump and sewage flow maintenance. Consequently, flows were adjusted to provide 1.5-2.0 gallons per minute for a 12-16 hour period from about 8 AM until 12 midnight, when a timer shut off the pump until sewage began flowing the next morning. Due to the difficulty of maintaining steady sewage flows without considerable operator maintenance, the facility was operated at irregular flow rates in between sampling periods in order to "feed" the biomass.

Sewage entering the facility was settled in a modified Imhoff conicle tank where it received approximately 1 hour retention time. The sludge was returned to the manhole. One week before sampling, and throughout each two week sampling period, sewage flows were carefully maintained at desired rates on a daily basis. Sampling operations were conducted approximately every 6 weeks for an 8 month period from September 1976 to March 1977. Although the fluctuating sewage influent conditions created an operational inconvenience, it actually presented a more difficult test for the treatment process than under consistent flows and nutrient loading conditions.

Operational Results

Facility start-up consisted of inoculation of the Bio-Web substrates with bacterial, protozoan, and invertebrate detritivore organisms cultured in our aquacultural food production systems, and stocking the tanks with water hyacinth and duckweed plants. Within 4 weeks time, a treatment effectiveness was readily observed, and the aquatic plants and snails were reproducing and growing most vigorously in the first cell with the high nutrient loading. As expected, the amphipod and ostracod invertebrates and small Gambusia (mosquito fish) were thriving better in the second, third and fourth cells, presumably due to the toxic levels of ammonia in the first cell. Detergent foams inhibited the growth of duckweeds in the first cell, but had little effect on the water hyacinths. Full flow conditions (1500 gallons a day) could be handled effectively within 3 months from start-up. The results of water chemistry analysis for the main wastewater parameters tested during the 6 month monitoring period from October 1976-March 1977 are presented in Figures 5a-7a. Analysis of BOD, suspended solids and coliforms was made by Environmental Engineering Laboratories, a certified testing laboratory. Nitrogen, phosphate, oxygen, pH, and temperature were analyzed by Solar AquaSystems personnel, with split samples verified by E. E. L.

The Bio-Web substrates were installed at densities of approximately 4.0 sq. ft. per gallon treated per day (6,000 sq. ft. of total surface area) in the first three tanks only. The fourth tank contained bio-film surface area in the form of plastic mesh cages for holding freshwater shrimp in individual cells at high density. The third tank contained five carp averaging 3 lbs each held within a net-pen. The purpose of the fish and shrimp was to observe their survival and growth within the system and to observe their ability to function as top trophic level consumers by cropping the invertebrate detritivores, which migrated steadily into their cages and pens.

A diverse biological film of diatom, some algae, and numerous bacterial and fungal slimes grew on the Bio-Web substrates in the first tank. In the following tanks, a much thinner film developed, consisting primarily of bacteria and diatoms which were well grazed by ostracods, protozoa, amphipods, and snail detritivores. During the eight month period the substrates were never observed to accumulate a film thicker than approximately 0.8 millimeters on either surface. A detritus or sludge layer that might be expected to accumulate on surfaces in conventional sewage systems did not build up. This is apparently due to the continual sloughing off of the vertically oriented and continuously flexing bio-film, as well as to grazing by the detritivore invertebrates. The sloughing off process appears to function independently of, and without need for, the grazing process, because during periods when the first cell contained few invertebrates, the bio-film layer did not increase.

Sludge build-up was monitored in the bottom of all four tanks. Results indicate that the anaerobic layer on the bottom of the first two cells was effective in reducing sludge build-up. After eight months operation, there remained only about 10 mm of sludge in Cell #1, 5 mm in Cell #2, and less than 4 mm of sediment in the remaining two cells.

The polyethylene greenhouse cover and the solar spraying heat transfer system operated effectively to heat the water during the winter, and cool the air during summer periods. Water temperatures could be increased on the average of 1.0°C per cell per day during the summer periods, and maintained equal to influent temperatures (approximately 21°C) during the fall months. During winter, water temperatures dropped an average of only 0.5°C per day, averaging 18°C influent, and 16°C during the coldest winter periods when external air temperatures averaged 10°C daily. Water treatment effectiveness did not reduce substantially during the winter periods as expected. Suspended solids and BOD treatment continued at efficiencies close to fall periods, but nitrogen removal rates decreased as anticipated due to decreased growth of aquatic plants and food chain organisms. Detailed analysis of winter period performance will be carried out with the new Cardiff facility.

CARDIFF AQUACELL DEMONSTRATION FACILITY

Design

Due to the limited sewage flow available at the Solana Beach site, a new

AquaCell was constructed at the San Elijo Treatment Facility in Cardiff, California. The facility, operated by the County of San Diego, provides primary treatment and ocean discharge of two million gallons per day of domestic sewage. At this new location either raw or primary treated effluent can be obtained dependably on a continuous basis, and the flow represents a more typical cross-section of municipal wastewater.

The Cardiff AquaCell facility is designed to approximate full-scale design parameters in terms of surface area to volume ratio, aeration rates, sewage loading rates, aquatic plant cover, etc. The AquaCell tanks were constructed as three cells in series, each 8' x 13' x 5 1/2' deep, and approximately 4,000 gallons capacity per cell. Design flow rates are from 2.0 to 4.0 gal/minute or 3,000 to 6,000 gal/day to allow 2-4 day retention time. Pretreatment will consist of a two-stage anaerobic tank (1,200 gallons capacity, Stage 1; 2,400 gallons capacity, Stage 2). The first stage is designed to function as the sedimentation and acid fermentation stage, and the second primarily as the methane fermentation stage, thereby improving treatment reliability, gas production, and sludge digestion. The anaerobic primary system has just recently been completed and operational data are not yet available.

The three AquaCell tanks are plumbed with header pipes at each end to distribute flows and minimize short-circuiting. The effluent and influent piping to each tank is plumbed to also allow either parallel or series flow operations, but to date has only been operated as series flow.

Cell #1 contains both diffused and surface aerators, while Cells #2 and 3 are aerated with only diffused aeration. The air diffusers are mounted approximately 1 foot off the bottom in order to minimize stirring of benthic sediments and to encourage the development of an anaerobic sediment digestion layer.

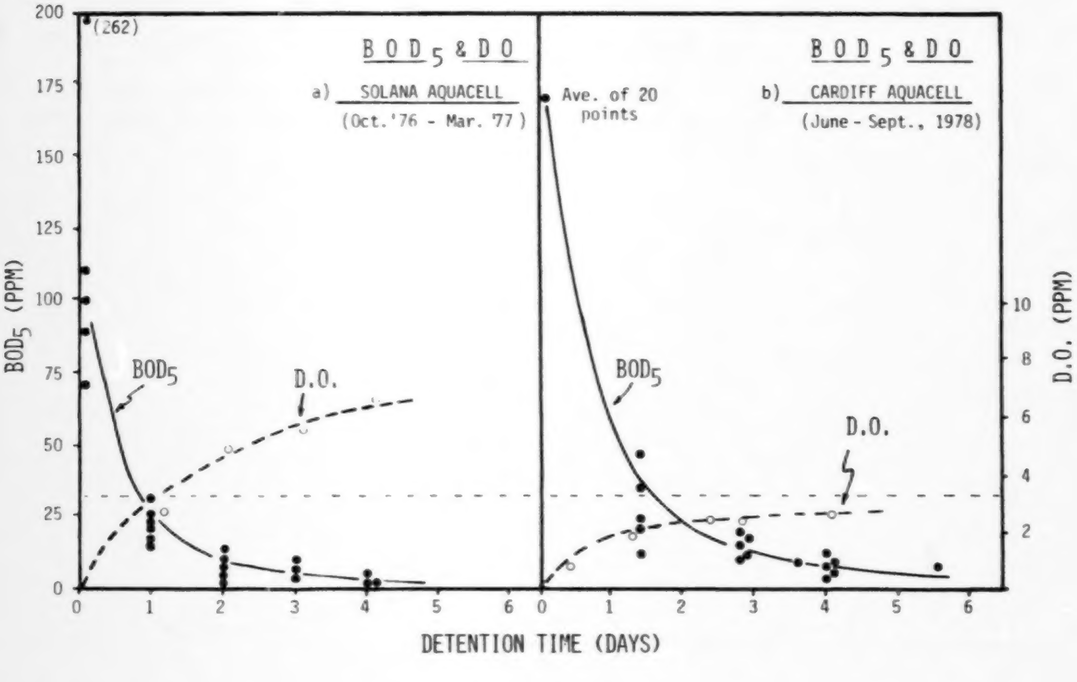


Figure 5a & b. Biological oxygen demand (BOD_5) and dissolved oxygen concentration in relation to detention time for Solana Beach and Cardiff Solar AquaCell demonstration facilities.

A slow-sand filter (0.1-0.2 gpm/sq. ft.) at the end of Cell #3 functions primarily as a screening device to remove invertebrate organisms, aquatic plant root fibers, and any re-suspended detritus. Sand particle size in the sand filter averages 1.0 mm, ranging from 0.5-2.0 mm in diameter. The sand filter has only required one back flushing during the first four months of continuous operation. Data presented in the following figures do not include sand filtration.

Biological Components

Biological components of the Cardiff AquaCell are similar to those of the Solana Beach facility. Aquatic plants consist of both water hyacinths and duckweeds (*Lemna minor*, *Spirodella spp.*, *Wolfia spp.*). The only fish stocked in the facility were *Gambusia* to control any mosquitos which might find their way into the tanks through open vents or doors. In order to demonstrate that the higher food chain organisms are not necessary for the treatment process, no other fish or shrimp were stocked during the first four months of operation. The Bio-Web substrates were installed at densities of 3 sq. ft. of total available surface area per gallon treated per day at the 3,000 gallon/day loading rate. The bio-film thickness was measured after three months operation and found to average 0.7 mm in Cell #1, 0.08 mm in Cell #2, 0.02 mm in Cell #3. The number of organisms per square millimeter was substantially greater in the first cell, but the diversity of microorganisms was greater in the second and third cells, as commonly observed in progressively cleaner zones of polluted streams (Hart and Fuller, 1974). Start-up operations were conducted for a one month period and consisted of inoculating the three cells with bacteria, sludge, detritus, microorganisms, amphipods, snails, and other detritivore invertebrates maintained in continuous culture in our aquaculture food production systems at a separate aquaculture facility.

Treatment Performance and Discussion

Results of chemistry analysis for the first four months operation indicate

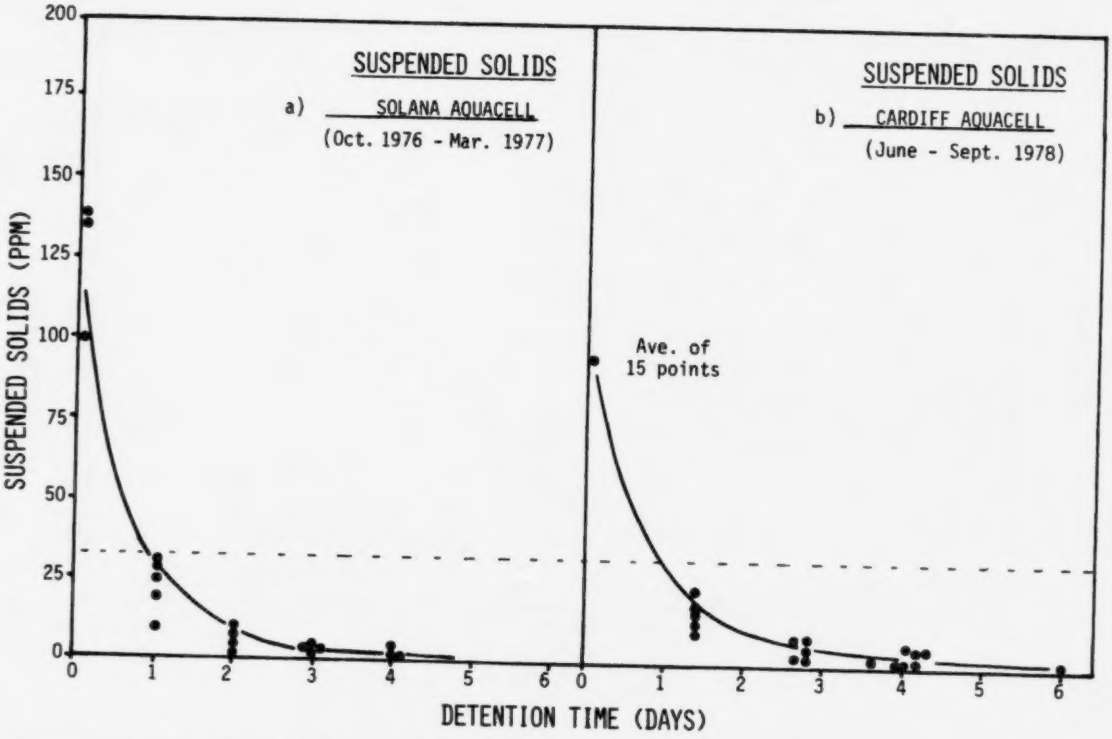


Figure 6a & b. Suspended solids treatment performance for Solana Beach and Cardiff solar AquaCell demonstration facilities.

treatment performance is similar to the Solana Beach facility. The chief differences are due to the influent sewage, which is substantially higher in BOD and ammonia at the Cardiff site. Even though treated by one hour sedimentation, the primary effluent is more typical of raw sewage and requires greater retention time than the relatively weaker sewage at the Solana Beach facility. This can be seen by comparing Figures 5a and 7a with 5b and 7b, showing the relative BOD and ammonia, nitrite, and nitrate levels over 4-6 days retention time. The BOD loading rate of 3,000 gpd at the Cardiff AquaCell has been equivalent to 1,800 lbs/acre/day in Cell #1 only, or an average of about 600 lbs/acre/day in all three cells. The higher BOD and ammonia loading rates also required considerably more aeration, which resulted in a water mixing rate too high to properly maintain the first cell as a facultative pond. Oxygen levels averaged 0.5 to 2.0 ppm in the upper half of the first cell, but dropped to only 0.5-1.0 ppm near the bottom. Preliminary observations of the system with the anaerobic stages in operation indicate the reduced BOD load permits reduced aeration, mixing, and allows more effective facultative operation in the first cell.

The high ammonia and BOD levels also reduce dissolved oxygen in Cell #2 and 3, thereby inhibiting nitrification, and causing higher BOD measurements at days 3 and 4 due to the remaining ammonia. (Nitrification of ammonia to nitrate requires 4.2 mg/O_2/mg/NH_3, or nearly four times the amount per mg of carbonaceous BOD.) Preliminary results with the anaerobic primary unit in operation indicates higher dissolved oxygen (3-4 ppm), lower ammonia, and lower BOD in Cells #2 and 3. Suspended solids have been steadily maintained at 2 to 8 ppm in final effluent after the start-up phase.

It is interesting to note that a secondary quality effluent could be achieved within two days retention time (1-1/2 cells), during which only the time in Cell #2 provided exposure to aquatic plants. It is clear from operational results, as well as existing bio-film type treatment processes, that

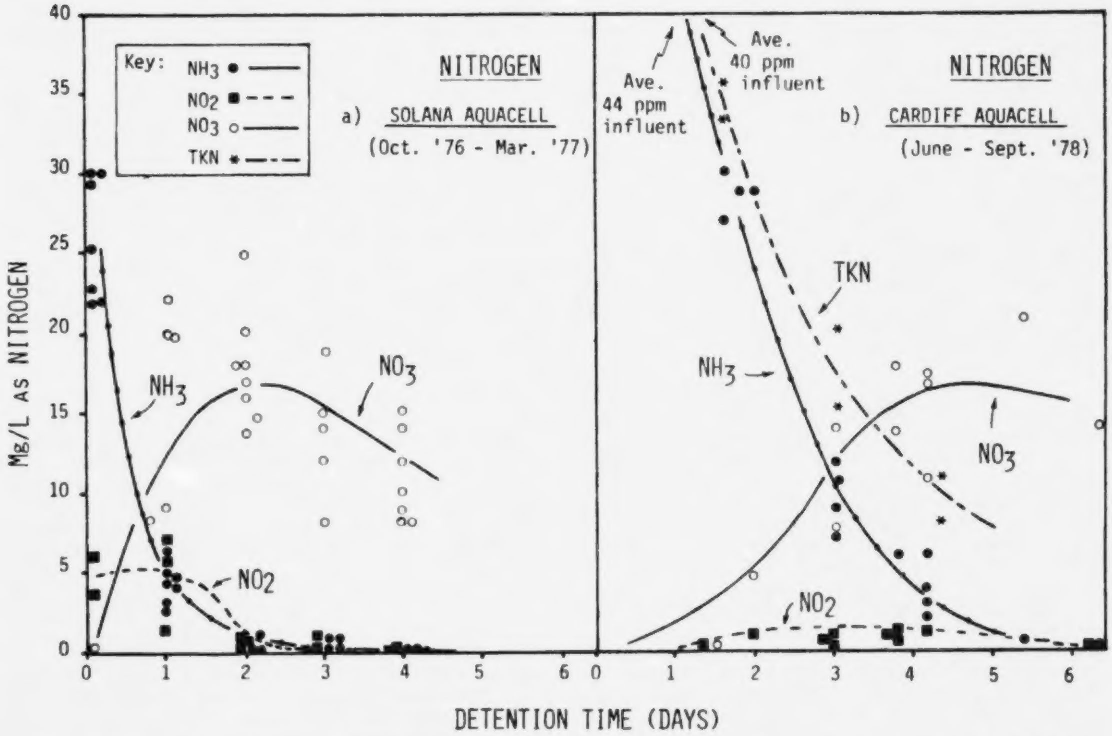


Figure 7a & b. Treatment and removal of ammonia, nitrite, nitrate, and total Kjeldahl nitrogen, expressed as mg/l nitrogen, for the Solana Beach and Cardiff AquaCell facilities.

secondary treatment can be achieved within short retention times using only the Bio-Web substrates. In contrast, use of only water hyacinths requires a minimum of 15-20 days (average year-round) to achieve secondary treatment (Wolverton, 1978; Del Fosse, 1977). The advantages of incorporating aquatic plants can be numerous, as previously discussed, but the plant species and method of use must be based on a thorough analysis of overall treatment and design objectives.

Effluent phosphate levels have been highly variable, due primarily to the widely fluctuating input levels (10-30 ppm) and short retention time masking the relatively low removal rates. Estimates based on limited data suggest averages of 0.5-1.0 ppm total phosphate removal per day retention time can be expected.

Total coliform reduction occurred rapidly (Figure 8) suggesting the need for only low doses of chlorine or ozone to meet most discharge criteria.

Total dissolved salt studies have not yet been carried out, awaiting completion of a relatively air-tight cover to approximate full-scale greenhouse operating conditions. Limited data from the Solana Beach AquaCell unit indicated reduction ranging from about 0-300 ppm (0-30 percent), but were highly variable. Dinges (1978) reported average TDS reductions up to 50 percent in a water hyacinth channel receiving secondary treated sewage. The biomass mineral uptake cannot account for more than 5-10 percent of the reduction Dinges or we observed. Humic acids are produced during the breakdown of aquatic macrophytes in the benthic layer. Chelation/humic colloid precipitation processes by humic acids are well known (Ruttner, 1963), and undoubtably account for this variable but potentially very useful process for TDS reduction.

During startup operations and first two months of steady flow, the sewage was entirely shut off for periods from 3-8 days on three different occasions. When sewage was returned to full flow, treatment effectiveness was reachieved within two days, attesting to the stability and elasticity of the bio-film and floating macrophyte plant process.

The data presented in Figures 5-7 suggest the functioning of the system is a fairly linear process, typical of a zero order reaction. The water quality improves in direct relation to retention time. This is in contrast to most lagoon processes which are subject to the fluctuating and often opposing processes of sedimentation, clarification, algae growth, die-off, nutrient release and regrowth, and so on, until eventual net stabilization of organic matter is gradually achieved after 50-150 days retention time. The rate of nitrification ranged from 0.1-0.3 lbs/day/1,000 sq. ft. of Bio-Web surface area, depending on dissolved oxygen levels (including the nitrification and/or ammonia removal by the hyacinths and duckweeds). This compares favorably with the 0.32-0.55 lbs/day/1,000 sq. ft. rate observed by Abd-El-Bary and Eways (1978) using fixed-films for nitrification.

Energy Considerations

Energy requirements of the AquaCell system are being monitored by use of air flow metering devices and by calculating electrical demand of the surface aerator and diffused air compressor. Modifications were made in the analysis to account for lack of efficiency with the small aeration devices used. Based on expected air volumes, transfer efficiencies, horsepower requirements for a full-scale system, the energy requirements for the present loading rates are equivalent to approximately 25 Hp per mgd. We estimate this power requirement can be reduced to 15-20 Hp per MGD with the anaerobic pretreatment process, depending on effluent quality desired.

In terms of energy requirements, the AquaCell process is similar to an aerated lagoon. Under conditions of high BOD, ammonia, and water temperature, it can require more aeration, primarily due to more complete nitrification and BOD reduction, and the production of advanced tertiary rather than secondary quality water. The preliminary results to date suggest the process requires considerably less energy than theoretical estimates, based on carbonaceous and nitrogenous BOD, would indicate.

As shown in Figure 4, aeration and oxygen demand are reduced by a number of natural processes, including: 1) direct uptake of ammonia and nutrients by the aquatic plants, which exchange oxygen and carbon dioxide through the air, thereby decreasing and stabilizing oxygen demand, particularly during nocturnal or low-sunlight hours; 2) direct entrapment and removal of particulate BOD by plant harvesting; 3) coagulation and sedimentation of organic suspended solids by the Bio-Web substrates, followed by anaerobic digestion on the pond bottom; 4) passive oxygen exchange through the water surface, assisted by gentle mixing from submerged diffused aerators; 5) no aeration requirements to suspend the microorganisms, since the Bio-Web sub-

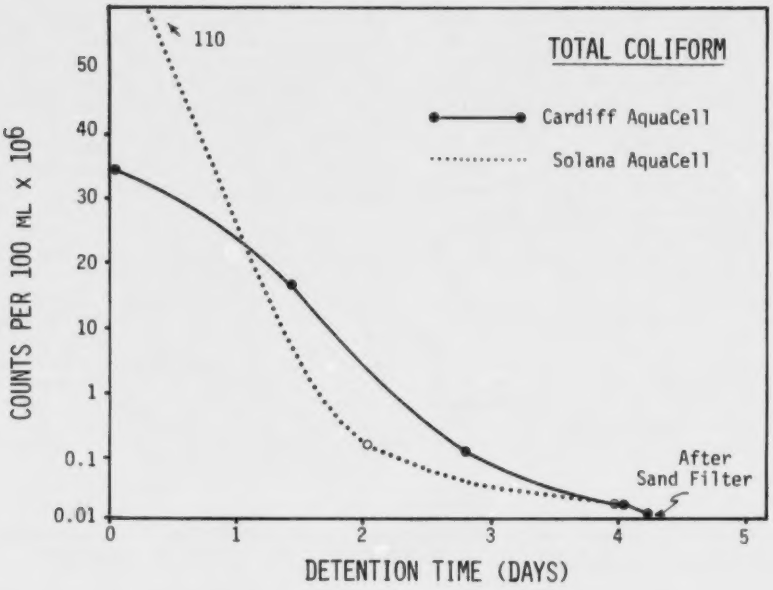


Figure 8. Average total coliform reduction in relation to detention time.

strates are buoyant and the ponds are operated only as a low-mix process; 6) the aquatic macrophytes and detritivore invertebrates convert nutrients into a form which remains stable until harvest. On-line or backup excess aeration is not required to provide for "upset" periods characteristic of algae or bacterial systems.

THE CITY OF HERCULES SOLAR AQUACELL ADVANCED TREATMENT FACILITY

The City of Hercules, 30 miles northeast of San Francisco, California, is presently constructing a 0.35 mgd, 1.5 acre lagoon treatment system using the Solar AquaCell Process. Upon successful operation of this first phase, the facility will be expanded to 6 acres of AquaCells for treating 2.0 mgd of raw, domestic sewage to advanced tertiary quality as the community grows over the next five to ten years. The facility is expected to begin start-up operations by March, 1979, approximately 5 months from start of construction.[1]

Facility Design

The Hercules facility is designed as a three phase system consisting of a two stage anaerobic pond, followed by a facultative lagoon, and a final third phase aerobic lagoon, as shown in Figure 9. In the ultimate facility, three independent facultative/aerobic pond units will be operated in parallel to allow the city flexibility to select and more efficiently meet changing water discharge or reuse needs. By varying the wastewater flow to either of the three ponds, detention time can be either increased to produce a higher quality effluent for industrial reuse, or decreased to produce a secondary quality effluent for land application or discharge. The flexibility provided by the AquaCell lagoon process was a major advantage desired by the city to allow process optimization under changing regulatory or environmental conditions, water availability, and reuse market demands over the next 30 years.

Pretreatment will consist only of grinding (comminutor) in order to minimize clogging problems during periodic removal of sludge from the anaerobic pond. The two stage anaerobic ponds are designed to provide approximately 20 hour total detention time, with the first stage designed to operate primarily as the sedimentation and acid fermentation stage, and the second stage as the methane fermentation stage. This will allow more control, stability and treatment efficiency of the anaerobic process. Both anaerobic ponds will be covered with floating hypalon type rubber covers with gas collection channels. The black covers will function as solar heat absorbers, and insulate against heat losses, in order to maintain higher operating temperatures and improve winter period treatment. The combination of aerobic with anaerobic ponds has the further advantage of balancing out seasonal differences in treatment efficiency between the ponds. The anaerobic phase will function better during the winter, due to improved sedimentation and reduced sludge digestion and thereby counteract the slightly decreased operating efficiency of the aerobic AquaCells during winter. During the summer periods, the reverse trend will occur. The increased sludge digestion in the anaerobic pond will decrease its effluent quality, while the facultative and aerobic AquaCells will produce higher quality effluent due to higher operating temperatures and increased aquatic plant and bacterial metabolism and growth.

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[1] The facility is 100 percent financed by the city and is designed jointly by Solar AquaSystems and KCA Engineers.

The facultative and aerobic treatment phases are combined in the same earthen pond, with a hypalon curtain wall hydraulically separating and providing a channel for the facultative treatment area. The hypalon curtain wall is integrated with the greenhouse columns, and therefore is relatively inexpensive to construct. The facultative stage is designed for approximately 24 hour retention time. Its function is threefold: 1) to

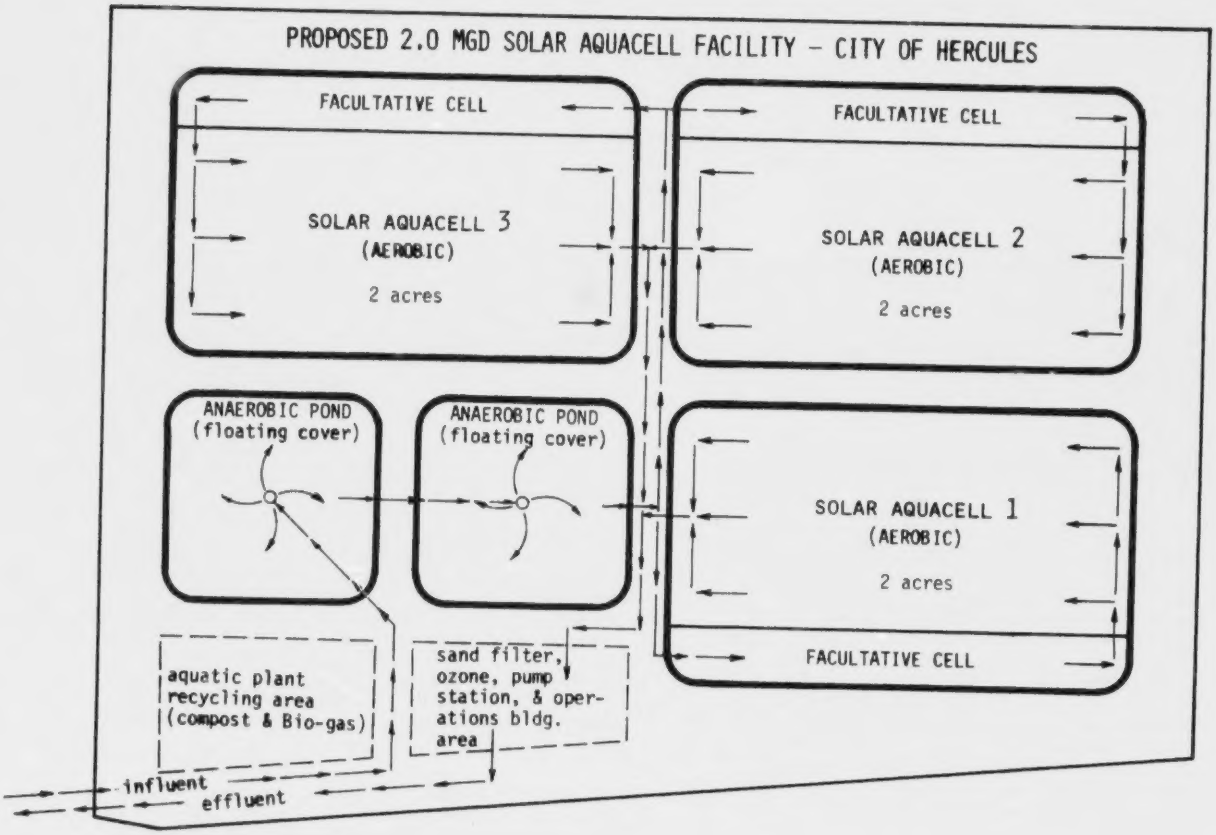
PROPOSED 2.0 MGD SOLAR AQUACELL FACILITY - CITY OF HERCULES
FACULTATIVE CELL
SOLAR AQUACELL 3
(AEROBIC)
2 acres
FACULTATIVE CELL
SOLAR AQUACELL 2
(AEROBIC)
2 acres
ANAEROBIC POND
(floating cover)
ANAEROBIC POND
(floating cover)
SOLAR AQUACELL 1
(AEROBIC)
2 acres
FACULTATIVE CELL
aquatic plant
recycling area
(compost & Bio-gas)
sand filter,
ozone, pump
station, & operations bldg.
area
influent
effluent

Figure 9. Plan view of the proposed 2.0 mgd Solar AquaCell Lagoon Treatment Facility for the City of Hercules, California. Each AquaCell will be 2.0 acres (6 acres total). The 0.35 mgd treatment phase currently under construction consists of a 1.5 acre AquaCell system with anaerobic, facultative and aerobic stages.

reduce BOD by use of higher aeration than will be maintained in the aerobic AquaCell stage; 2) to reduce BOD by means of coagulation and sedimentation of suspended solids via the Bio-Web substrate mechanism, and; 3) to nitrify ammonia in order to reduce toxicity in the following aerobic stage. The facultative pond will not contain water hyacinths or duckweeds, in order to maximize oxygen exchange through the surface. Diffuser pipes at the influent and effluent ends of the facultative and aerobic ponds minimize short circuiting problems.

In the aerobic Solar AquaCell pond, oxygen levels will be maintained from 2 ppm at the influent to 6 ppm or greater at the effluent end. Water hyacinths and duckweeds will be cultured over the entire water surface of this stage in order to prevent growth of algae, to assist the Bio-Web reduction of BOD, SS, total nitrogen and phosphates, and to produce a valuable by-product. Multiple overflow screens will prevent escape of detritivore invertebrates.

Projected Effluent Quality

Final effluent will pass through a sand filter and ozone contact chamber to assure advanced tertiary effluent for industrial reuse purposes. Ozone treatment was selected over chlorine, since only relatively low doses of ozone will be required with the low BOD, SS, and ammonia effluent levels. Ozone treatment can also have additional benefits by oxidizing organic chemicals and preventing the formation of toxic chlorinated hydrocarbons which would be caused by use of chlorine.

If additional phosphate removal is required for certain reuse purposes, chemical addition could be minimal because of the reduced interference from BOD, SS, and ammonia in the effluent. With the quality of the water projected, phosphate removal costs should be about one-third of normal requirements.

For the northern California climate, we estimate that approximately 1.5 acres of Solar AquaCells (3 day retention time) are required for 1.0 mgd flow (about 12,000 people) to achieve secondary effluent quality. Advanced tertiary (less than 5 ppm BOD, SS, less than 10 ppm total nitrogen, and insignificant levels of ammonia, heavy metals and chlorinated hydrocarbons) will require approximately 6 days retention time or 6 acres for 2.0 mgd.

Water and Aquatic Plant By-Product Reuse

The high quality effluent will allow multiple reuse options for the city. Two local industries have expressed interest in purchasing the water for process operations. Greenbelt and forage crop irrigation needs exist within the community, and two creeks, often dry through most of the year, are suitable for enhancement or incorporation into a marsh-wildfowl sanctuary.

The aquatic plants will be harvested on a weekly or monthly basis, depending on seasonal productivity, and converted into organic compost in the first years of operation. Digested sludge from the primary treatment pond will be removed approximately once every four to six months, or as required, to be composted together with the harvested plants in order to improve the sludge texture and dilute any possible concentrations of heavy metals to levels safe for garden use. In the final facility, the possibility of converting all the aquatic plants and sludge to methane gas will be feasible, with the methane used to operate the air compressors and reduce total energy and operating costs.

Because of the success experienced to date by Dr. George Allen (Allen et al., 1977) with culturing salmon fingerlings on natural food organisms grown in wastewater lagoons, this possibility is being considered for the Hercules project. Salmon fingerlings would be cultured in the net pens within the AquaCell ponds for release down the creek enhanced by the reclaimed water, out to the Pacific Ocean, and to return 2-3 years later as a private salmon run.

ACKNOWLEDGMENTS

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INTERMITTENT SAND FILTRATION TO UPGRADE LAGOON EFFLUENTS

James H. Reynolds, Jerry Russell, and E. J. Middlebrooks*

INTRODUCTION

Nature of the Problem

Wastewater stabilization lagoons provide simple, efficient, and economical wastewater treatment for over 4,000 communities throughout the United States. Approximately 90 percent of these wastewater stabilization lagoon systems serve communities having a population of less than 5,000 persons. These small communities are often lacking in resources and competent personnel to operate and maintain sophisticated wastwater treatment facilities.

Historically, wastewater stabilization lagoons have provided adequate wastewater treatment for these small communities, however, wastewater lagoon effluent may not meet present and future stringent wastewater discharge standards. In order to meet these stringent standards, an inexpensive treatment method which does not require sophisticated operation and maintenance and one which is economical is needed for upgrading or polishing lagoon effluents.

Intermittent sand filtration is capable of polishing lagoon effluents at relatively low cost. Intermittent sand filtration is not a new technique. Rather, it is the application of an old technique to the problem of upgrading lagoon effluents. Intermittent sand filtration is similar to the practice of slow sand filtration in potable water treatment or the slow sand filtration of raw sewage which was practiced during the early 1900s. Intermittent sand filtration of lagoon effluents is the application of lagoon effluent on a periodic or intermittent basis to a sand filter bed. As the wastewater passes through the sand filter bed, suspended solids and organic matter are removed through a combination of physical straining and biological degradation processes. The particulate matter collects in the top 5 to 7.5 cm (2 to 3 inches) of the sand filter bed. This buildup of organic matter eventually clogs the top 5 to 7.5 cm (2 to 3 inches) of the sand filter bed and prevents the passage of the effluent through the sand filter. The sand filter is then taken out of service and the top layer of clogged sand is removed. The sand filter is then put back into service and the spent sand is either discarded or washed and used as replacement sand for the sand filter.

Objectives

The objective of this paper is to discuss the use of intermittent sand filtration to polish or upgrade wastewater stabilization lagoon effluent. Emphasis will be placed on intermittent sand filtration research conducted by Utah State University.

A historical perspective and review of intermittent sand filtration has been compiled by Hill et al. (1976) and Harris et al. (1977).

SINGLE STAGE FILTRATION

The development of intermittent sand filtration to upgrade lagoon effluents has been conducted primarily at Utah State University (Marshall and Middlebrooks, 1974; Reynolds et al., 1974a; Harris et al., 1975; Reynolds et al., 1974b; Hill et al., 1977; Messinger et al., 1977; Bishop et al., 1976; Tupyi et al., 1977). The work at Utah State University has been conducted on laboratory scale, pilot scale, and field scale filters. However, the principal work was conducted on six prototype scale filters shown in Figure 1. Each of these filters was operated at a different hydraulic loading rate for 12 months. The filter sand employed in the study was upgraded pit run sand with an effective size of 0.17 mm (0.007 inch) and a uniformity coefficient of 9.74 (see Table 1).

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The biochemical oxygen demand (BOD_5) removal performance for each of the six filters is shown in Figure 2. The overall average influent BOD_5 concentration was 19 mg/l. The influent BOD_5 concentration ranged from 3.5 mg/l to over 288 mg/l, exceeding 5 mg/l, 94 percent of the time. Figure 2 shows the consistently high quality of filter effluent, which was unaffected by influent BOD_5 fluctuations. Effluent quality was below 5 mg/l 93 percent of the time (except filter number 2 during the winter). The effluent BOD_5 con-

Table 1. Sieve analysis of filter sand[a] (Reynolds et al., 1974).

U.S. Sieve Designation Number	Size of Opening (mm)	Percent Passing (%)
3/8"	9.5	100.0
4	4.76	92.1
10	-	61.7
40	0.42	27.0
100	0.149	6.2
200	0.074	1.7

[a] e = 0.170 mm; u = 9.74.

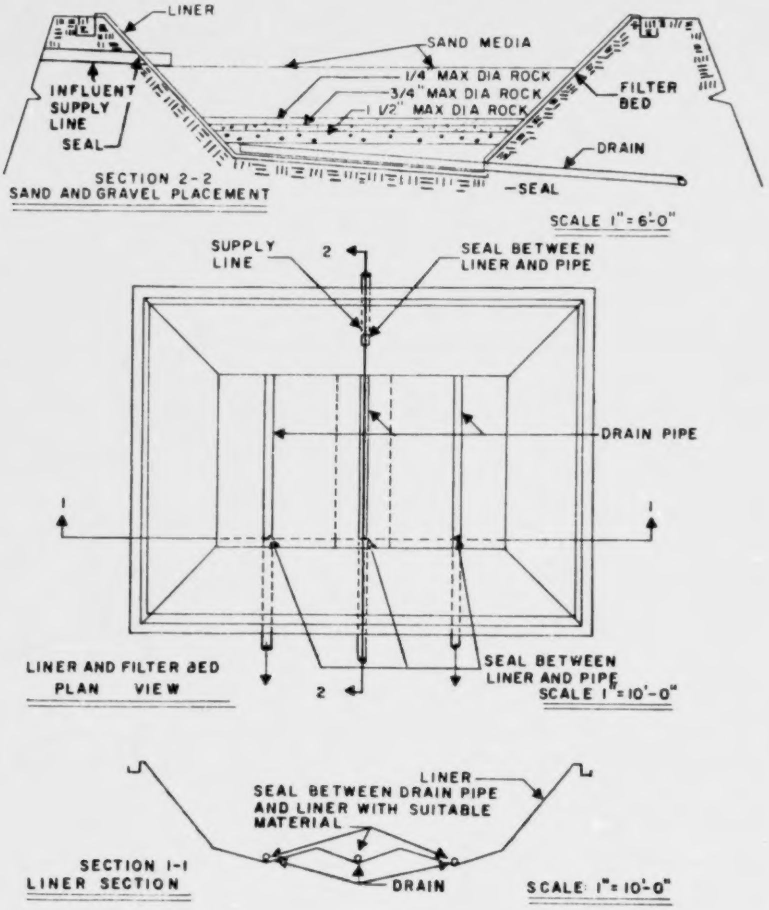

LINER
SAND MEDIA
1/4 MAX DIA ROCK
3/4" MAX DIA ROCK
1 1/2" MAX DIA ROCK
INFLUENT SUPPLY LINE
SEAL
FILTER BED
DRAIN
SEAL
SECTION 2-2
SAND AND GRAVEL PLACEMENT
SCALE 1" = 6'-0"
SUPPLY LINE
2
SEAL BETWEEN LINER AND PIPE
DRAIN PIPE
1
1
LINER AND FILTER BED
PLAN VIEW
2
SEAL BETWEEN LINER AND PIPE
SCALE 1" = 10'-0"
LINER
SEAL BETWEEN DRAIN PIPE AND LINER WITH SUITABLE MATERIAL
SECTION 1-1
LINER SECTION
DRAIN
SCALE 1" = 10'-0"

Figure 1. Cross section of a typical intermittent sand filter (Harris et al., 1975).

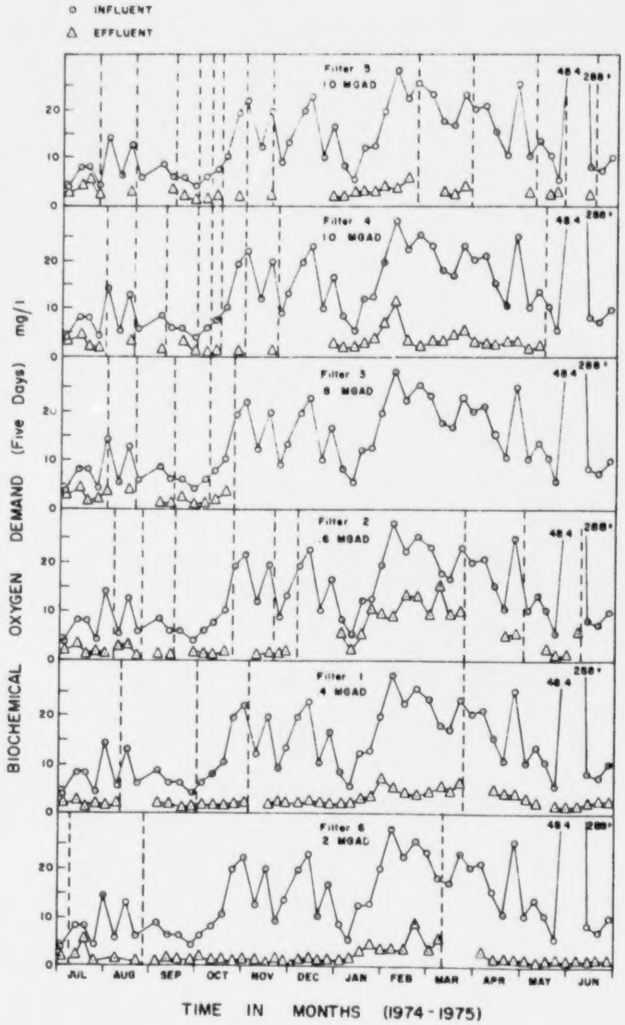


Figure 2. Single stage intermittent sand filtration biochemical oxygen demand (BOD_5) removal performance.

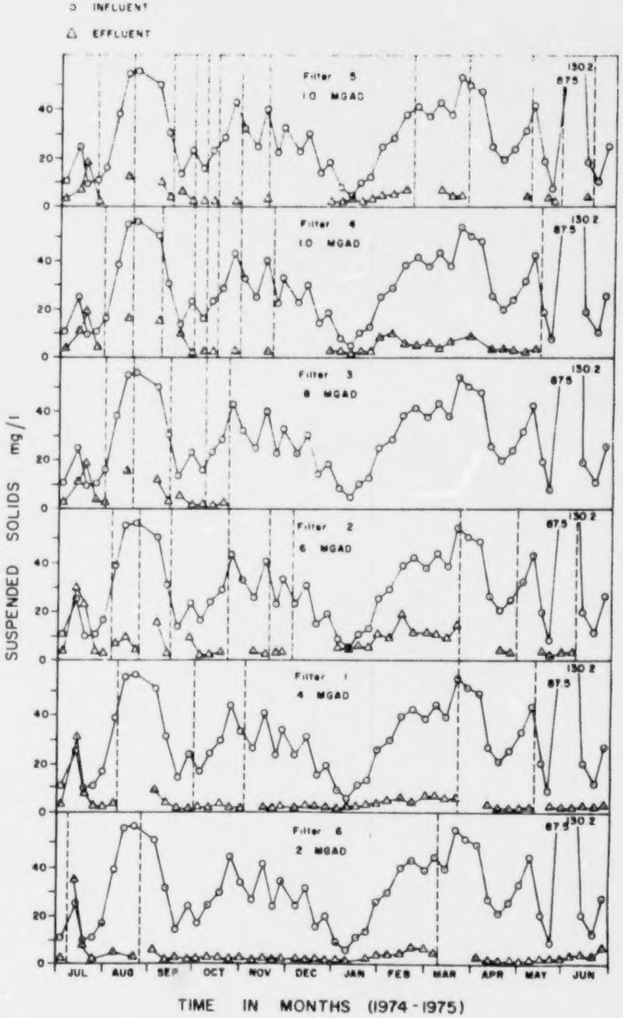


Figure 3. Single stage intermittent sand filtration suspended solids removal performance (Harris et al., 1975).

centration never exceeded 11.8 mg/l and exceeded 7 mg/l only four times during the study.

The winter operation of filter number 2 was different from the other filters. Filter number 2 was operated in a constant flooded mode during winter operation. This constant flooded mode was an attempt to improve winter operation. However, the anaerobic condition that developed due to this constant flooding greatly reduced the filter efficiency. The effluent BOD_5 concentration for filter number 2 exceeded 5 mg/l 92 percent of the time. At the end of the winter operation, filter number 2 was returned to normal operation and as a result the filter effluent returned to normal when compared to the other filters in the study.

Suspended solids removal performance of the filters is shown in Figure 3. The influent exceeded 5 mg/l suspended solids concentration, 100 percent of the time and was greater than 30 mg/l 44 percent of the time. The average influent suspended solids concentration was 31 mg/l with a low of 5.5 mg/l and a high of 130.2 mg/l. The three filters in operation at the time of maximum influent suspended solids (130.2 mg/l) had effluent suspended solids concentrations of less than 3 mg/l.

The initial loading period (Figure 3) shows a relatively high concentration of suspended solids in the filter effluent. This is primarily due to the "washing" of fine dirt (inorganic fine) from sand during the start-up period. After this initial start-up period, the filter effluent exceeded 5 mg/l suspended solids only 15 percent of the time and had averages of less than 6 mg/l. Seldom did the effluent exceed 10 mg/l. The anaerobic conditions of filter number 2 during winter operations caused its effluent to exceed 5 mg/l 83 percent of the time. The suspended solids concentrations from each filter were consistently low.

The length of filter runs achieved during this study was a function of the hydraulic loading rate and the influent suspended solids concentration. Filter run lengths ranged from 8 days at a hydraulic loading rate of 9,360 m^3/ha.d (1.0 mgad) during the summer months to 188 days at a hydraulic loading rate of 1,872 m^3/ha.d (0.2 mgad) during winter months.

Intermittent sand filters should be similar in design to those illustrated in Figure 1 with hydraulic loading rates of 2,744 to 5,616 m^3 ha.d (0.4 to 0.6 mgad). Filter sands should have an effective size of 0.15 to 0.25 mm (0.006 to 0.010 inch) with a uniformity coefficient (Fair et al., 1968) from 1.5 to 10. The expected filter run length from intermittent sand filters designed on the above criteria will depend on the filter influent quality, but should be a minimum of 30 to 60 days.

These results (Harris et al., 1975; Reynolds et al., 1974; Reynolds et al., 1975) clearly indicate that intermittent sand filtration of lagoon effluents can produce a final effluent with a BOD_5 concentration and a suspended solids concentration of less than 15 mg/l consistently throughout the entire year even during periods of sub-zero temperatures.

EFFECT OF SAND SIZE ON SINGLE STAGE PERFORMANCE

Tupyi et al. (1977) studied the effect of various size filter sands on intermittent sand filter performance. The work employed the same filter system as Harris et al. (1975) (see Figure 1) except the effective size of the filter sand employed for the various filters was 0.68 mm, 0.40 mm, 0.31 mm, and 0.17 mm. Hydraulic loading rates ranging from 1871 m^3/ha.d (0.2 mgad) to 28,061 m^3/ha.d (3.0 mgad) were studied. In addition, application rates ranging from 0.008 m^3/sec (0.29 cfs) to 0.048 m^3/sec (1.74 cfs) were investigated.

The biochemical oxygen demand (BOD_5) performance of all the intermittent sand filters with respect to various effective size sands, hydraulic loading rates and application rates is recorded in Table 2 and illustrated in Figure 4. Yearly average BOD_5 concentration in the effluent applied to the filters was 11 mg/l with the daily BOD_5 concentration ranging from 3 mg/l to 22 mg/l throughout the study.

The BOD_5 removal performance of the 0.40 mm and 0.68 mm effective size sand (Filters No. 2, 3, 4, and 5) with a high application rate of 0.048 m^3/sec (1.68 cfs) was not adequate to produce an effluent that was consistently less than 10 mg/l. The 0.31 mm effective size sand (Filter No. 3) produced a significant BOD_5 removal; however, the influent characteristics during this phase of study indicate that these results were inconclusive. Lowering the application rate on the 0.40 mm effective size sand (Filter No. 5) and the 0.68 mm effective size sand (Filter No. 4) appeared to increase BOD_5 removal; how-

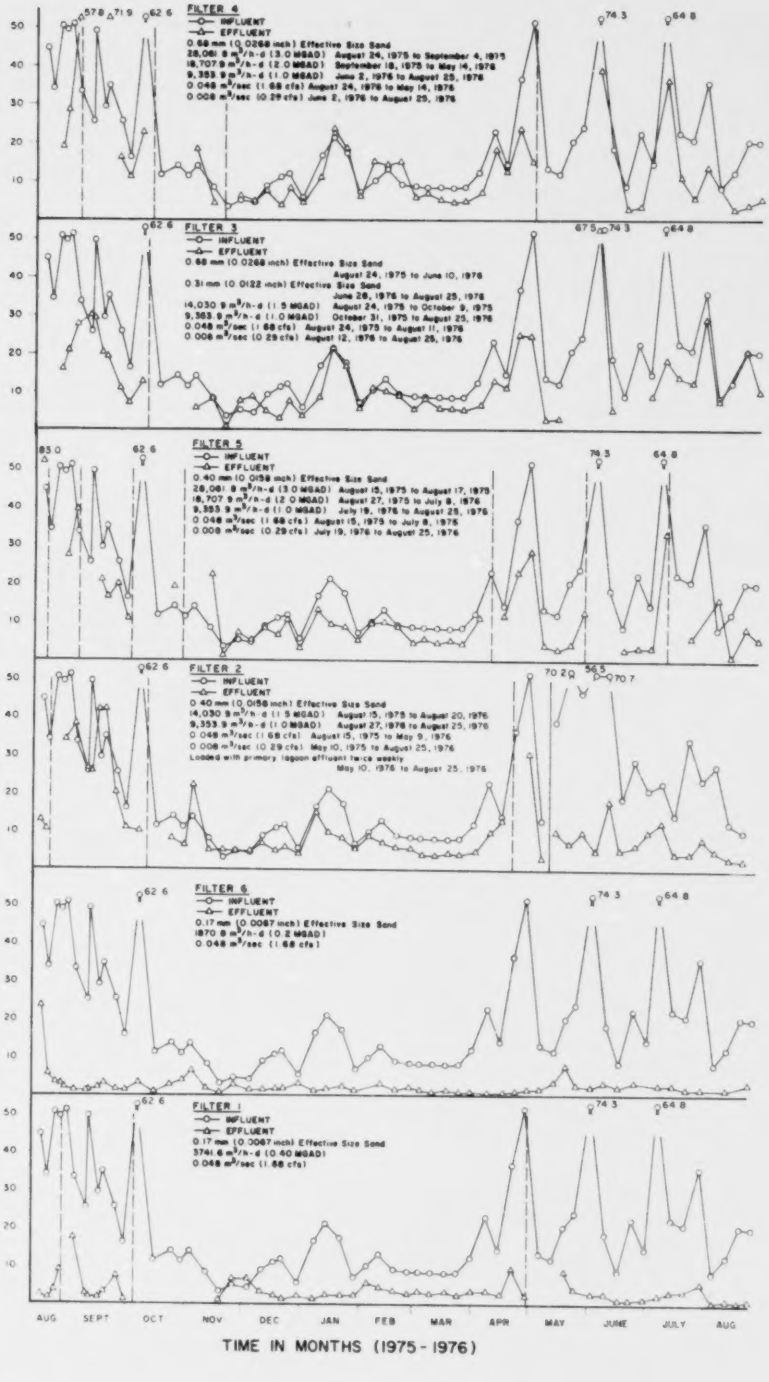


Figure 5. The weekly suspended solids performance.

Table 2. A summary of the 5-day biochemical oxygen demand performance.

Effective Size Filter Sand (mm)	Hydraulic Loading Rate ($m^3/h.d$)	Application Rate (m^3/sec)	Influent BOD^5 (mg/l)			Effluent BOD^5 (mg/l)			Average Percent Removal
			Min.	Max.	Ave.	Min.	Max.	Ave.	
0.17	1,870.8	0.048	2.6	22.3	10.9	0.3	3.7	1.1	90.1
0.17	3,741.6	0.048	2.6	22.3	11.5	0.1	7.4	2.6	77.2
0.31	9,353.9	0.048	4.5	20.8	13.8	5.0	11.2	7.8	43.5
0.31	9,353.9	0.008	9.7	9.7	9.7	6.1	6.1	6.1	33.7
0.40	9,353.9	0.048	2.6	22.3	10.5	3.7	17.8	8.2	21.9
0.40	9,353.9	0.008	4.5	19.8	12.2	3.7	11.0	5.4	56.0
0.40	14,030.9	0.048	9.8	11.5	10.7	3.9	6.0	5.0	53.3
0.40	18,707.9	0.048	2.6	22.3	11.3	2.6	23.3	8.6	23.9
0.40	28,061.9	0.048	9.8	9.8	9.8	4.5	4.5	4.5	54.6
0.68	9,353.9	0.048	2.6	22.3	11.6	4.1	17.3	8.2	28.8
0.68	9,353.9	0.008	3.7	20.8	13.2	2.9	15.4	7.9	39.8
0.68	14,030.9	0.048	4.4	12.9	7.8	4.0	6.8	5.7	27.5
0.68	18,707.9	0.048	2.6	22.3	11.5	3.2	15.6	9.0	21.0
0.68	28,061.9	0.048	6.3	12.9	9.4	2.9	11.9	6.8	27.1
Loaded With Primary Lagoon Effluent Twice Weekly									
0.40	9,353.9	0.008	9.1	75.6	26.8	3.7	28	10.5	60.8

ever, the zooplankton in the influent during that experiment make such a conclusion questionable. The 0.17 mm effective size sand (Filters No. 1 and 6) was shown to be capable of high BOD_5 removal at low hydraulic loading rates of 3742 $m^3/ha \cdot d$ (0.4 mgad) and 1871 $m^3.ha \cdot d$ (0.2 mgad). No conclusion can be established with relation to the Federal Secondary Treatment Standards which requires an effluent BOD_5 of 30 mg/l or less because the influent BOD_5 concentration did not exceed 23 mg/l during the entire study period.

Suspended solids (SS) removal by intermittent sand filters with various effective size filter sands, hydraulic loading rates, and application rates are shown in Table 3 and Figure 5. The mean yearly influent suspended solids concentration (secondary lagoon effluent) was 23 mg/l and the daily SS concentration ranged 3 mg/l to 65 mg/l.

The 0.68 mm, 0.40 mm, and the 0.31 mm effective size sand (Filters 2, 3, 4, and 5) with a high application rate of 0.048 m^3/sec (1.68 cfs) were unable to satisfy a 10 mg/l standard more than 50 percent of the time. Lowering the application rate to 0.008 m^3/sec (0.29 cfs) on the 0.68 mm and 0.40 mm effective size sand filters (Filters No. 4 and 5) increased suspended solids removal performance and meeting a 10 mg/l standard a minimum of 67 percent of the time. The indication that influent suspended solids significantly influence effluent suspended solids concentrations preclude the use of these filter sands to satisfy stringent discharge standards. It appears that lower application rates increase SS removal, but a definite conclusion cannot be reached due to the short period of study at the lower application rates and the heavy growth of Daphnia in the secondary lagoon effluent during the low application rate study.

The 0.40 mm effective size sand (Filter No. 2) with a hydraulic loading rate of 9,354 $m^3/ha.d$ (1.0 mgad) and a low application rate of 0.008 m^3/sec (0.29 cfs) loaded with primary lagoon effluent twice weekly produced high SS removals. Suspended solids removal averaged 76 percent during the study and further indicates that application rate may have a definite effect on SS removal. However, operation of this filter does not represent normal single stage intermittent sand filter operation since lagoon effluent was applied to the filter only twice weekly, rather than daily.

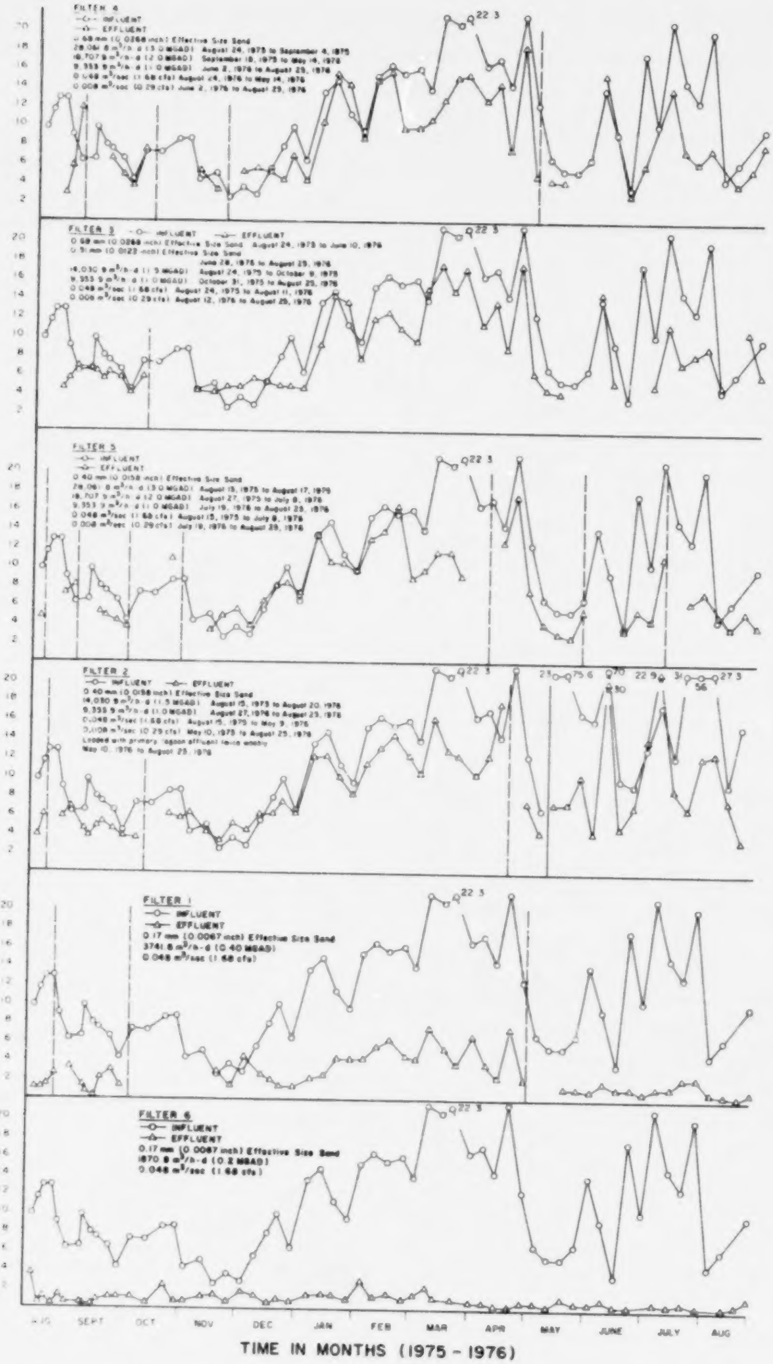


Figure 4. The weekly 5-day biochemical oxygen demand performance.

Table 3. A summary of the suspended solids performance.

Effective Size Filter Sand (mm)	Hydraulic Loading Rate (m^3/ha.d)	Application Rate (m^3/sec)	Influent SS (mg/l)			Effluent SS (mg/l)			Average Percent Removal
			Min.	Max.	Ave.	Min.	Max.	Ave.	
0.17	1,870.8	0.048	3.2	74.3	23.0	0.6	23.8	2.7	88.2
0.17	3,741.6	0.048	3.2	74.3	20.8	0.3	17.6	3.5	83.0
0.31	9,353.9	0.048	8.2	64.8	27.8	7.7	29.2	15.4	44.6
0.31	9,353.9	0.008	20.0	20.1	20.1	10.2	21.0	15.6	22.4
0.40	9,353.9	0.048	3.2	51.5	18.7	1.2	30.8	13.1	30.1
0.40	9,353.9	0.008	12.2	35.9	21.8	1.8	16.0	7.5	65.5
0.40	14,030.9	0.048	34.3	44.9	39.6	10.7	12.5	11.6	70.7
0.40	18,707.9	0.048	3.2	64.8	18.1	1.2	39.8	11.6	35.9
0.40	28,061.9	0.048	44.9	44.9	44.9	83.0	83.0	83.0	0
0.68	9,353.9	0.048	3.2	51.5	15.8	1.6	25.2	11.2	29.3
0.68	9,353.9	0.008	8.9	74.3	34.1	2.9	39.7	15.5	54.7
0.68	14,030.9	0.048	17.7	51.6	38.2	7.5	30.0	19.6	48.6
0.68	18,707.9	0.048	3.2	51.5	16.7	3.2	24.4	13.1	21.6
0.68	28,061.9	0.048	33.2	51.6	44.9	19.3	57.8	35.4	21.1
Loaded With Primary Lagoon Effluent Twice Weekly									
0.40	9,353.9	0.008	10.5	70.7	34.0	3.2	18.0	7.9	76.7

The 0.17 mm effective size sand (Filters No. 1 and 6) with hydraulic loading rates of 3742 m^3/ha·d (0.4 mgad) and 1871 m^3/ha·d (0.2 mgad) are capable of meeting the 10 mg/l standard and the Federal Secondary Discharge Standard of 30 mg/l.

A summary of filter run lengths with the various size sands is shown in Table 4. High hydraulic loading rates of 28,061.9 m^3/ha·d (3.0 mgad) produce undesirable short filter run lengths for 0.40 mm (0.0158 inch) and 0.68 mm (0.0268 inch) effective size sand filters (Filters No. 2, 3, 4, and 5). Hydraulic loading rates of 18,707.9 m^3/ha·d (2.0 mgad) and less produce satisfactory filter run lengths for the 0.40 mm (0.0158 inch) and 0.68 mm (0.0268 inch) effective size sand filters (Filters No. 2, 3, 4, and 5). The 0.17 mm 0.0067 inch) effective size sand filter (Filter No. 6) with a hydraulic loading rate of 1870.8 m^3/ha·d (0.2 mgad) did not plug during the entire one year study. The 0.17 mm (0.0067 inch) effective size sand filter (Filter No. 1) with a hydraulic loading rate of 3741.6 m^3/ha·d (0.4 mgad) produced satisfactory filter run lengths. Due to insufficient data of the 0.31 mm (0.0122 inch) effective size sand filter (Filter No. 3) and the 0.68 mm (0.0268 inch) and 0.40 mm (0.0158 inch) effective size sand filters (Filters No. 4 and 5) with hydraulic loading rates of 9353.9 m^3 ha·d (1.0 mgad) and a low application rate of 0.008 m^3/sec (0.29 cfs), no conclusion can be reached. However, data collected thus far suggest that filter run length may be increased by lowering the application rate.

SERIES INTERMITTENT SAND FILTRATION

In an attempt to increase the length of filter run achievable with intermittent sand filtration of lagoon effluent, Hill et al. (1977) conducted laboratory and pilot scale studies on a series arrangement of intermittent sand filters. The experimental design employed in these studies is shown in Figure 6. Series intermittent sand filtration involves the passage of lagoon effluent through two or three intermittent sand filters arranged in series with progressively smaller effective size sands. Hill et al. (1977) investigated three different hydraulic loading rates on a pilot scale basis.

The weekly BOD_5 removal performance of the pilot scale series inter-

Table 4. Filter run lengths achieved by the various effective size sand filters during the experimental period.

Effective Size Sand (mm)	Hydraulic Loading Rate (m^3/ha·d)	Application Rate (m^3/sec)	Suspended Solids Removal (kg)	Volatile Suspended Solids Removal (kg)	Method of Rejuvenation	Consecutive Days of Operation
0.17	1,870.8	0.048	121.03	100.17	N.A.	280 and 94
0.17	3,741.6	0.048	14.19	10.26	Scraped	11
0.17	3,741.6	0.048	29.69	22.65	Scraped	36
0.17	3,741.6	0.048	55.95	53.47	Scraped and Rested 14 days	166
0.17	3,741.6	0.048	75.56	68.68	N.A.	103
0.31	9,353.9	0.048	44.43	48.29	N.A.	45
0.31	9,353.9	0.008	5.45	15.02	N.A.	14
0.40	9,353.9	0.048	40.92	63.31	Scraped	44
0.40	9,353.9	0.048	59.10	60.08	Scraped	177
0.40	9,353.9	0.048	20.47	19.73	N.A.	17
0.40	9,353.9	0.008	42.06	39.41	N.A.	37
0.40	14,030.9	0.048	20.03	17.31	Scraped	6
0.40	18,707.9	0.048	15.25	20.26	Scraped	7
0.40	18,707.9	0.048	28.33	37.35	Rested 22 days	18
0.40	18,707.9	0.048	0.00	2.86	Scraped	6
0.40	18,707.9	0.048	68.00	67.77	Scraped	148
0.40	18,707.9	0.048	87.01	65.71	Scraped	42
0.40	18,707.9	0.048	61.98	57.34	Scraped	23
0.40	28,061.9	0.048	0.00	17.89	Scraped	3
0.68	9,353.9	0.048	71.67	79.46	N.A.	196
0.68	9,353.9	0.008	124.20	98.16	N.A.	84
0.68	14,030.9	0.048	102.57	102.03	Scraped	46
0.68	18,707.9	0.048	51.31	42.43	Rested 19 days	23
0.68	18,707.9	0.048	0.00	11.93	Scraped	19
0.68	18,707.9	0.048	101.26	106.82	Scraped	152
0.68	18,707.9	0.048	14.95	12.85	N.A.	11
0.68	28,061.9	0.048	46.36	47.22	Scraped	11
Loaded with Primary Lagoon Effluent Twice Weekly						
0.40	9,353.9	0.008	62.25	72.74	N.A.	30

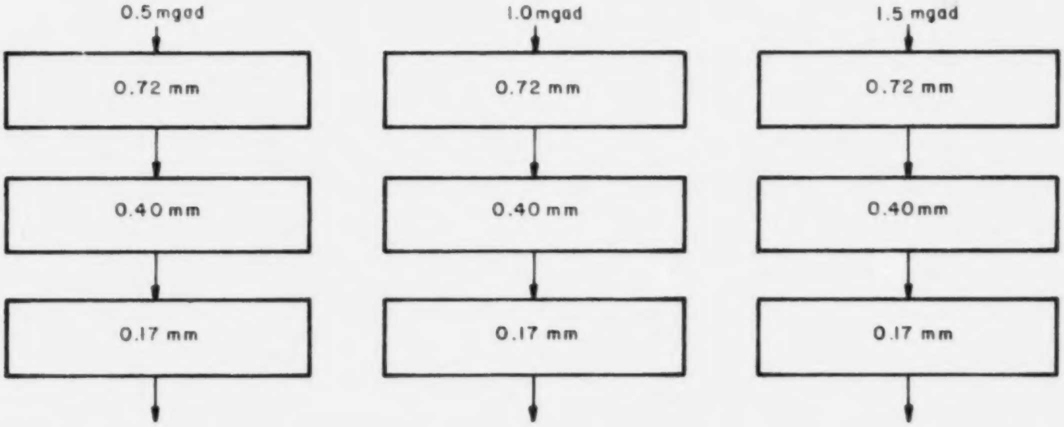


Figure 6. Series intermittent sand filtration of lagoon effluents (Hill et al., 1977).

mittent sand filters is illustrated in Figure 7. The influent BOD_5 concentration varied from 4.1 mg/l to 24.0 mg/l and averaged 10.7 mg/l during the study. The final effluent BOD_5 concentration varied from 0.6 mg/l at the 14,031 m^3/ha·d (1.5 mgad) hydraulic loading rate to 4.2 mg/l at the 9,353 m^3/ha·d (1.0 mgad) hydraulic loading rate. At no time did the final effluent BOD_5 concentration from the operation exceed 5.0 mg/l. Statistical analysis at the 1 percent level revealed that the effluent BOD_5 concentration was statistically identical at all three hydraulic loading rates for the 0.72 mm (0.0284 inch) and 0.40 mm (0.0158 inch) effective size sand filters. The effluent BOD_5 concentration from the 0.17 mm (0.0067 inch) effective size sand filter at the 14,031 m^3/ha·d (1.5 mgad) hydraulic loading rate was significantly higher (1 percent level) than from the 9,353.9 m^3/ha·d (1.0 mgad) and 4,677 m^3/ha·d (0.5 mgad) hydraulic loading rates. However the actual numerical differences were small.

The weekly suspended solids removal performance of the series pilot acale field filters is shown in Figure 8. The influent suspended solids concentration ranged from 12.5 mg/l to 69.4 mg/l and averaged 32.4 mg/l for the study. The final average effluent suspended solids concentration ranged from 8.6 mg/l for the 4,677 m^3/ha·d (0.5 mgad) hydraulic loading rate to 6.4 mg/l for the 14,031 m^3/ha·d (1.5 mgad) hydraulic loading rate. There was no significant difference (1 percent level) between the final effluent suspended solids concentrations among the different hydraulic loading rates.

Figure 8 indicates that at the beginning of the study, the final effluent suspended solids concentration was dependent upon the influent concentration. This was due to the "filter

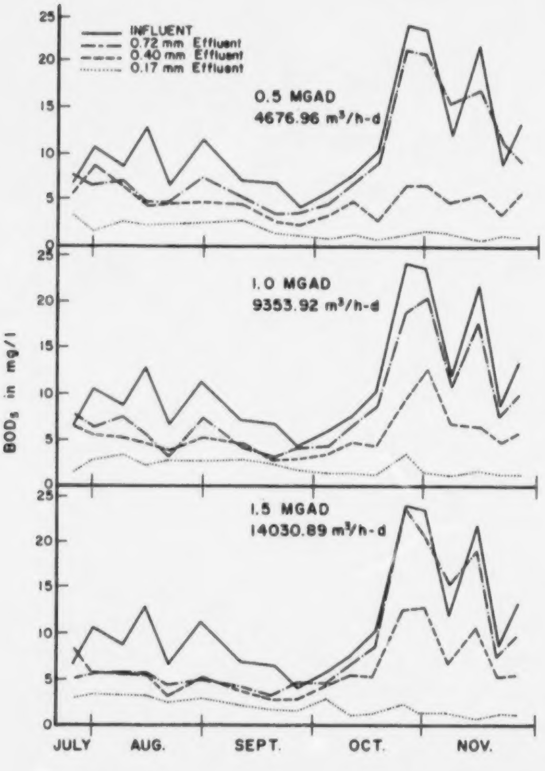


Figure 7. Weekly BOD_5 removal performance of pilot scale series intermittent sand filtration (Hill et al., 1977).

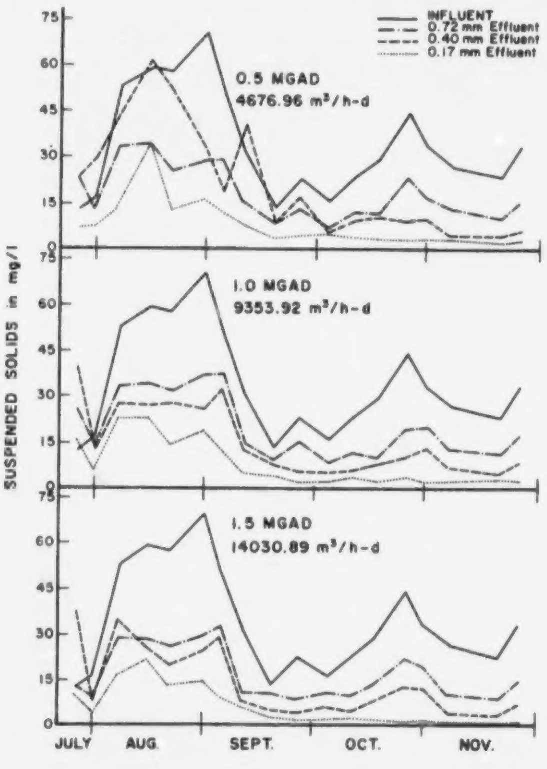
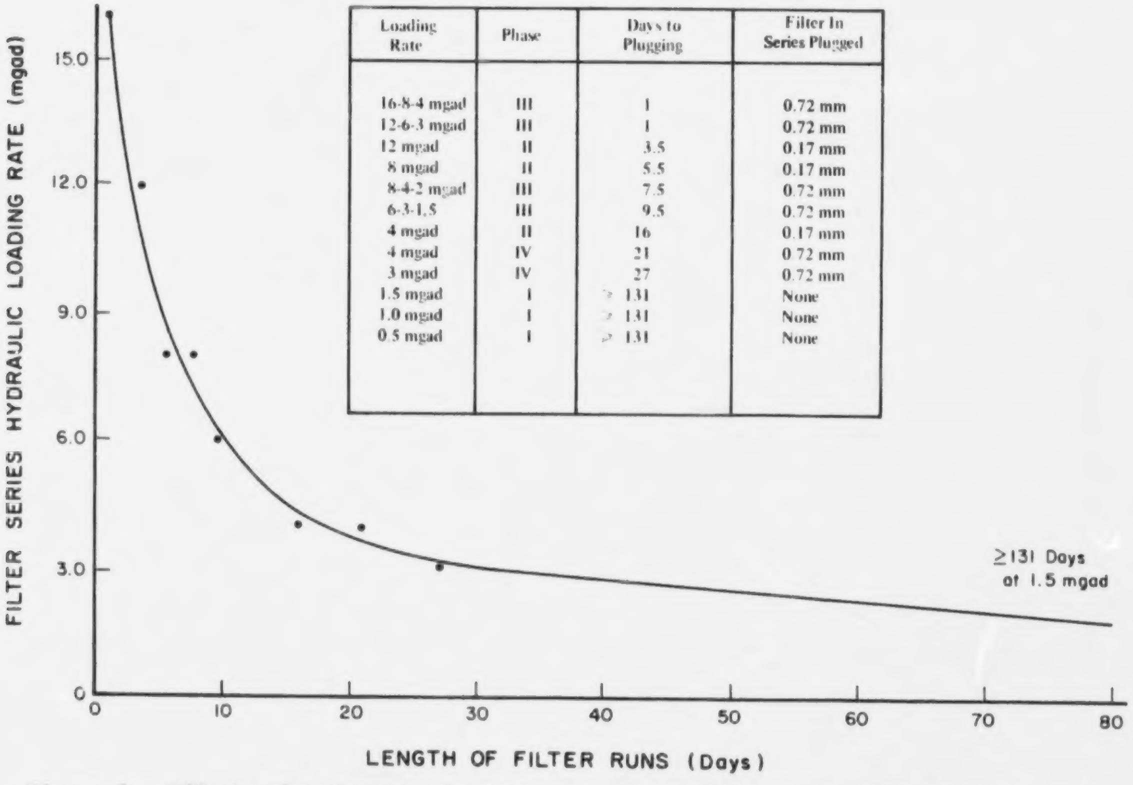


Figure 8. Weekly suspended solids removal performance of pilot scale series intermittent sand filtration. (Hill et al., 1977).

washing" effect which takes place during the initial start-up of intermittent sand filters when inert fines must be washed from the filter. When this washing was completed, excellent removals were obtained. At the end of the experimental phase, when removals were exceptional, the 0.17 mm (0.0067 inch) effective size sand filter effluent suspended solids concentration was essentially independent of the influent concentration. The efficiency of removal of intermittent sand filters is increased as the "schmutzdecke" (filtering skin) builds up on the surface of the filters.

One of the main advantages obtained with the use of a series intermittent sand filtration operation is the increased length of the filter runs. At the time operations were suspended due to freezing conditions (December 2, 1974), all three filter systems had operated for 131 consecutive days without plugging. Until the time operations ceased, the applied influent loading passed completely through all three filters in the series within 4 hours. It is difficult to estimate the length of filter run which could have resulted if freezing had not occurred; however, based on the data available, filter runs of at least 131 days may be obtained with a hydraulic loading between 4,677 m3/ha·d (0.5 mgad) and 14,031 m3/ha·d (1.5 mgad).

Hill et al. (1977) conducted a series of both laboratory and pilot scale studies of series intermittent sand filtration to determine the effect of hydraulic loading rates on the length of filter run achievable with three stage series intermittent sand filtration. The results are summarized in Figure 9. The results indicate that hydraulic loading rates greater than 28,080 m3/ha·d (3.0 mgad) significantly reduce the length of filtration run. However, Figure 9 neglects the effect of influent suspended solids concentration on filter run length. Thus, variations to Figure 9 will occur as the influent suspended solids concentration varies.



Loading Rate	Phase	Days to Plugging	Filter In Series Plugged
16-8-4 mgad	III	1	0.72 mm
12-6-3 mgad	III	1	0.72 mm
12 mgad	II	3.5	0.17 mm
8 mgad	II	5.5	0.17 mm
8-4-2 mgad	III	7.5	0.72 mm
6-3-1.5	III	9.5	0.72 mm
4 mgad	II	16	0.17 mm
4 mgad	IV	21	0.72 mm
3 mgad	IV	27	0.72 mm
1.5 mgad	I	≥ 131	None
1.0 mgad	I	≥ 131	None
0.5 mgad	I	≥ 131	None

Figure 9. Effect of hydraulic leaching rate on a three-stage series intermittent sand filtration (Hill et al., 1977).

The results of these studies indicate that a three stage series intermittent sand filtration system should be designed with filter sands of effective sizes between 0.72 mm (0.0284 inch) and 0.17 mm (0.007 inch) arranged according to Figure 6. Hydraulic loading rates should not exceed 28,080 m^3/ha·d (3.0 mgad) and preferably should be in the 14,031 m^3/ha·d (1.5 mgad) range. Using this criteria, filter run lengths in excess of 131 days should be possible.

Filtration of Aerated Lagoon Effluent

Bishop et al. (1976) conducted a pilot scale single stage intermittent sand filtration study to determine the feasibility of upgrading aerated lagoon effluent with intermittent sand filters. The results of the study clearly indicate that although BOD_5 removal was acceptable (i.e., effluent concentrations less than 30.0 mg/l) suspended solids concentrations were not significantly reduced. Thus, direct intermittent sand filtration of aerated lagoon effluent does not appear feasible.

The study (Bishop et al., 1976) also investigated the single stage intermittent sand filtration of a facultative lagoon which was preceded by an aerated lagoon. The results indicated that a high quality effluent (less than 20 mg/l BOD_5 and SS) can be obtained. Thus, aerated lagoon effluent may be upgraded utilizing intermittent sand filtration provided that the aerated lagoon is followed by a facultative lagoon before the effluent is applied to the intermittent sand filtration.

Intermittent Sand Filtration of Anaerobic Lagoon Effluent

A laboratory scale study was conducted to determine the feasibility of intermittent sand filtration of anaerobic lagoon effluent (Messinger et al., 1977; Bishop et al., 1976). The results indicated that direct intermittent sand filtration of anaerobic lagoon effluent is not feasible. The filter effluent BOD_5 and suspended solids concentrations were substantially greater than 30.0 mg/l in this study.

CASE STUDIES

Description

Three full scale lagoon-intermittent sand filter systems have been studied for three separate 30 day periods from January 1977 to June 1978. The three systems located at Mt. Shasta, California, Moriarty, New Mexico, and Ailey, Georgia, are shown schematically in Figures 10, 11, and 12 and described in Table 5.

Performance

A summary of the overall performance of each system is reported in Table 6.

The BOD_5 concentration of the three systems is illustrated in Figures 13, 14, and 15. At Mount Shasta, California, the mean filter influent BOD_5 concentration was 22 mg/l while the mean filter effluent BOD_5 concentration was 11 mg/l with a range of 2 mg/l. During Tour No. 1 (Mount Shasta) abnormally high filter effluent BOD_5 concentrations are probably a result of short circuiting through the filter caused by frozen filters and excessively high hydraulic loading rates. Proper filter operation will eliminate problems due to freezing (Harris et al., 1977).

At Moriarty, New Mexico, mean filter effluent BOD_5 concentrations were 20 mg/l and 21 mg/l during Tours No. 1 and 3, respectively. However, during Tour No. 2, the mean filter effluent BOD_5 concentration was reduced to 10 mg/l. This reduction was due to proper filter maintenance and lower influent BOD_5 concentrations.

Table 5. Design criteria for full scale systems at Mount Shasta, California, Moriarty, New Mexico, and Ailey, Georgia.

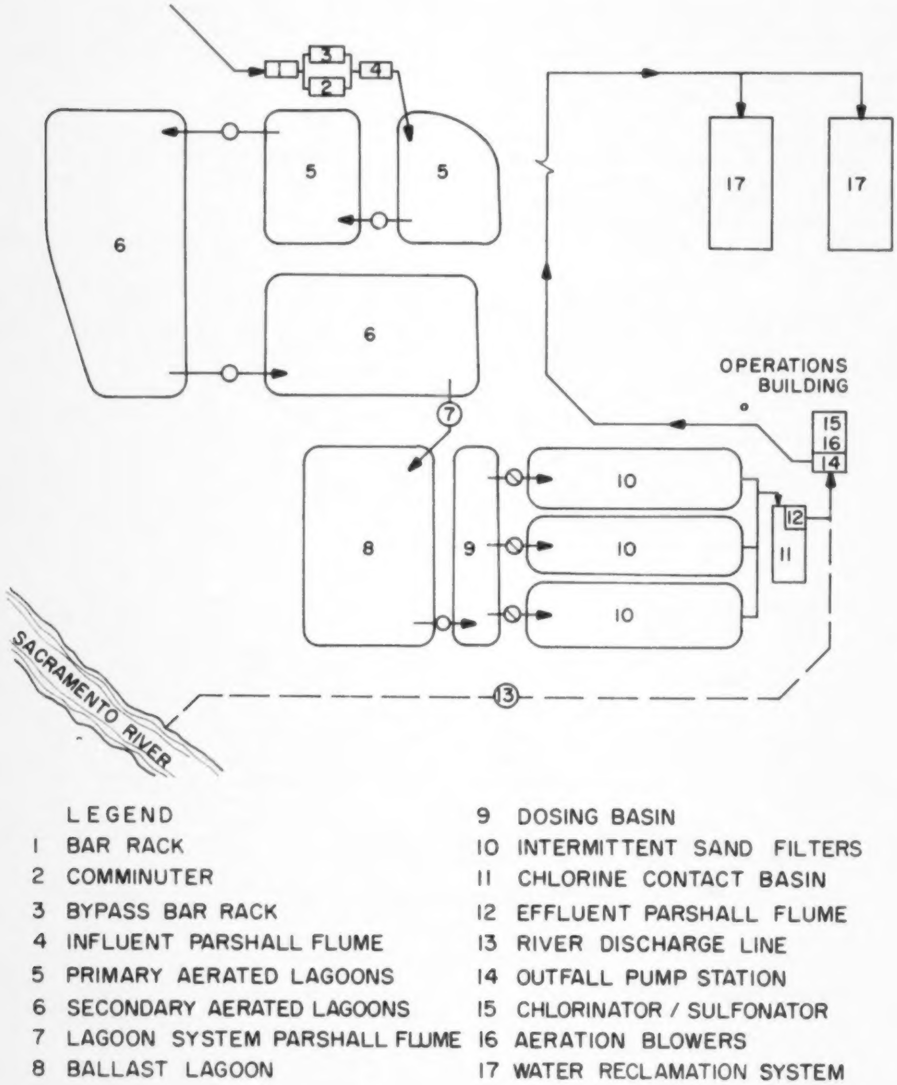
Parameter	Mount Shasta	Moriarty	Ailey
Design Q (mgd)	1.2	0.4	0.08
Lagoon Type	Aerated	Aerated/Facultative	Facultative
Filter Area (acre)	0.5	0.082	0.14
No. Filters	6	8	2
Hydraulic L.R. (mgad)	0.7	0.6	0.4
Effective Size (mm)	0.37	0.20	0.50
Uniformity Coeff.	5.1	4.1	4.0

At Ailey, Georgia, the mean filter effluent BOD_5 concentration was 8 mg/l with a range of 2 mg/l to 22 mg/l. This system produced a consistently high quality effluent.

The suspended solids performance for the three systems is illustrated in Figures 16, 17, and 18. At Mount Shasta, California, the three tours produced a mean filter effluent suspended solids concentration of 17 mg/l with a range of 1 mg/l to 49 mg/l. The high values are associated with the winter freezing problem.

At Moriarty, New Mexico, the three tour mean filter effluent suspended solids concentration was 13 mg/l with a range of 2 mg/l to 39 mg/l. The mean filter influent suspended solids concentration was 81 mg/l.

At Ailey, Georgia, the three tour mean filter effluent suspended solids concentration was 15 mg/l with a range of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SACRAMENTO RIVER
LEGEND
1 BAR RACK
2 COMMINUTER
3 BYPASS BAR RACK
4 INFLUENT PARSHALL FLUME
5 PRIMARY AERATED LAGOONS
6 SECONDARY AERATED LAGOONS
7 LAGOON SYSTEM PARSHALL FLUME
8 BALLAST LAGOON
9 DOSING BASIN
10 INTERMITTENT SAND FILTERS
11 CHLORINE CONTACT BASIN
12 EFFLUENT PARSHALL FLUME
13 RIVER DISCHARGE LINE
14 OUTFALL PUMP STATION
15 CHLORINATOR / SULFONATOR
16 AERATION BLOWERS
17 WATER RECLAMATION SYSTEM

Figure 10. Schematic of Mt. Shasta lagoon-intermittent sand filter system.

1 mg/l to 45 mg/l. The mean filter influent suspended solids concentration was 55 mg/l.

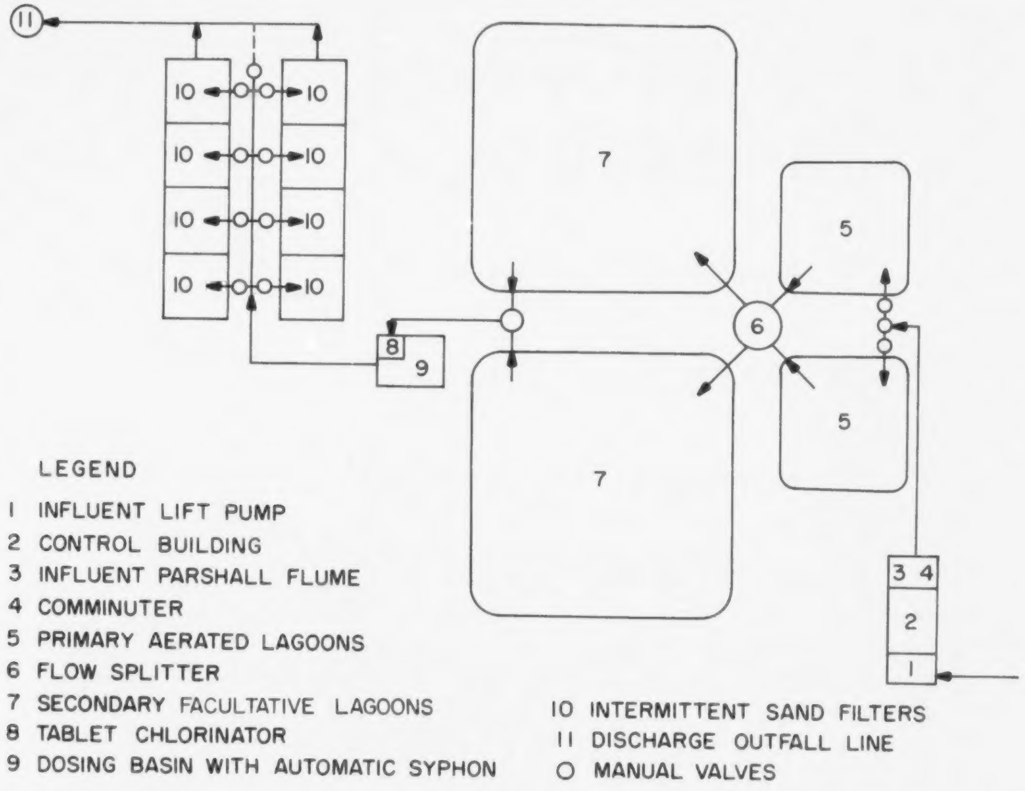
Operation and Maintenance

Maintenance requirements for the three sites were basically the same, except for the increase in requirements for the Mt. Shasta facility due to its complex nature and larger size. Each of the sites used the same basic processes that required common routine maintenance to provide a normal design service life.

Table 7 presents a summary of the reported maintenance requirements for each of the three sites for a period of approximately one year. The Moriarty, New Mexico, and Ailey, Georgia, report is the most complete and the most representative of requirements for maintenance of a lagoon-intermittent sand filter system.

The Mt. Shasta facility was seemingly plagued with problems either induced or accidentally caused by the operator. These problems many times resulted in extensive repairs and in some cases complete overhaul of portions of the system. The aeration system at Mt. Shasta also was a source of many problems observed at the facility. Constant operational changes and problems in conjunction with the extensive maintenance activities have distorted the reported maintenance requirements for the Mt. Shasta facility.

In general, each of the three full scale lagoon-intermittent sand filter systems consistently produced an effluent BOD_5 and suspended solids concentration of less than 30 mg/l. Reported maintenance requirements varied with the skill and ability of the operator. However, operation and maintenance of lagoon-intermittent sand filter systems appears to be less costly than constant systems.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LEGEND
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2 CONTROL BUILDING
3 INFLUENT PARSHALL FLUME
4 COMMINUTER
5 PRIMARY AERATED LAGOONS
6 FLOW SPLITTER
7 SECONDARY FACULTATIVE LAGOONS
8 TABLET CHLORINATOR
9 DOSING BASIN WITH AUTOMATIC SYPHON
10 INTERMITTENT SAND FILTERS
11 DISCHARGE OUTFALL LINE
○ MANUAL VALVES

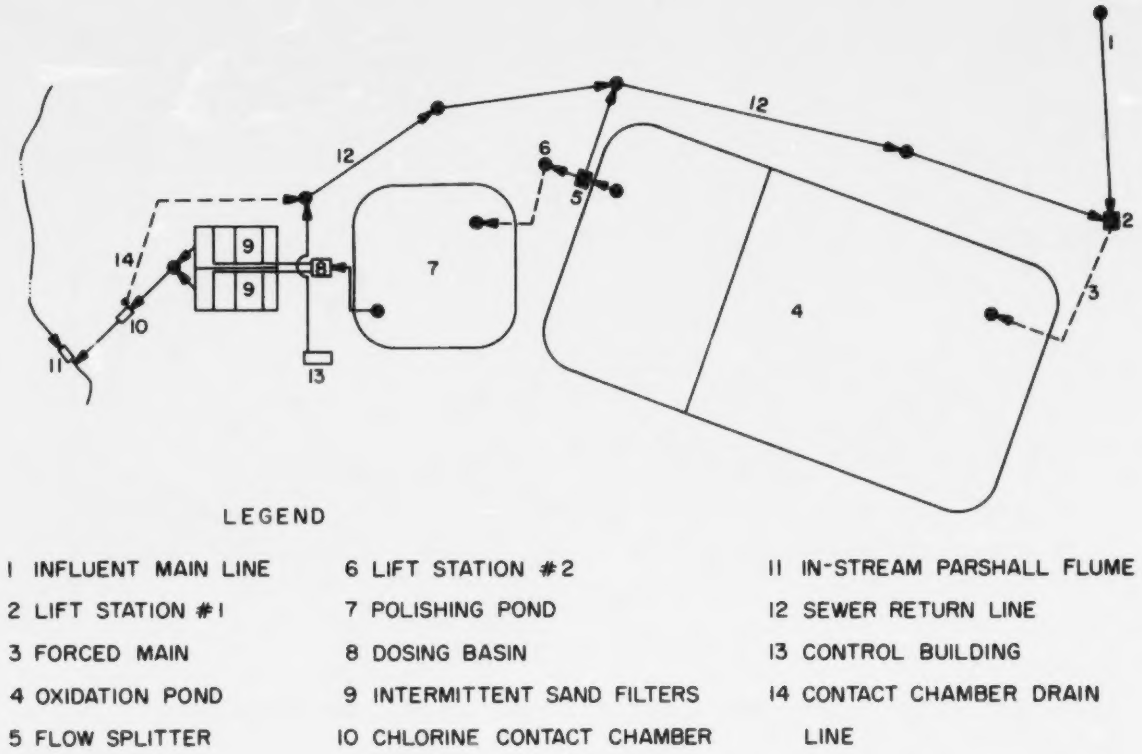
Figure 11. Schematic of Moriarty, New Mexico, lagoon-intermittent sand filter system.

Table 6. Summary of composite mean values for each sample point at each sample site during three tours.

	Mt. Shasta Water Pollution Control Facility				Moriarty Wastewater Treatment Facility				Ailey Sewage Treatment Plant			
Parameter	Facility Inf.	Lagoon Eff.	Filter Eff.	Facility Eff.	Facility Inf.	Lagoon Eff.	Filter Eff.	Facility Eff.	Facility Inf.	Lagoon Eff.	Filter Eff.	Facility Eff.
BOD (mg/1)	114	22	11	8	148	30	17	17	67	22	8	6
S. BOD (mg/1)	41	7	4	5	74	17	16	16	17	10	6	5
S.S. (mg/1)	83	49	18	16	143	81	13	13	109	43	15	13
V.S.S. (mg/1)	70	34	13	10	118	64	9	9	87	32	8	6
F.C. (col/100m)	1.16×10^6	292	30	<2	4.24×10^6	290	18	34	2.17×10^6	55	8	<1
I-pH (pH)	6.9	8.7	6.8	6.6	8.0	8.9	8.0	8.0	7.3	8.9	7.1	6.8
I-DO (mg/1)	4.8	12.4	5.5	5.3	1.8	10.9	8.3	8.3	6.7	10.2	7.4	7.9
COD (mg/1)	244	100	87	68	305	84	43	43	160	57	32	25
S. COD (mg/1)	159	71	64	50	197	67	34	34	82	41	23	16
Akl (mg/1 as $CaCO_3$)	95	75	51	42	436	293	260	260	93	84	76	69
TP (mg-P/1)	4.68	3.88	3.09	2.72	10.3	4.02	2.8	2.8	4.96	3.10	2.67	2.45
TKN (mg-N/1)	15.5	11.1	7.5	5.2	60	22	12.1	12.1	14.2	7.3	4.1	2.2
NH_3(mg-N/1)	10.8	5.56	1.83	1.76	38	16	9.16	9.16	5.5	0.658	0.402	0.31
Org-N (mg-N/1)	4.8	5.6	5.7	3.4	22	5.7	3.3	3.3	8.7	6.7	3.8	1.9
NO_2 (mg-N/1)	0.16	0.56	0.077	0.020	0.05	0.159	1.66	1.66	0.479	0.028	0.073	0.010
NO_3 (mg-N/1)	0.28	0.78	4.3	4.5	0.05	0.09	4.09	4.09	1.6	0.15	2.36	2.14
Total Algal Count (cells/ml)	N.A.	398022	144189	141305	N.A.	756681	32417	32417	N.A.	349175*	21583*	29360*
Flow	0.637	N.A.	N.A.	0.488	0.096	N.A.	0.046	N.A.	N.A.	N.A.	N.A.	0.070

N.A. = Not Available

* = For Tours #1 and #2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LEGEND
1 INFLUENT MAIN LINE
2 LIFT STATION #1
3 FORCED MAIN
4 OXIDATION POND
5 FLOW SPLITTER
6 LIFT STATION #2
7 POLISHING POND
8 DOSING BASIN
9 INTERMITTENT SAND FILTERS
10 CHLORINE CONTACT CHAMBER
11 IN-STREAM PARSHALL FLUME
12 SEWER RETURN LINE
13 CONTROL BUILDING
14 CONTACT CHAMBER DRAIN LINE

Figure 12. Schematic of Ailey, Georgia, lagoon-intermittent sand filter system.

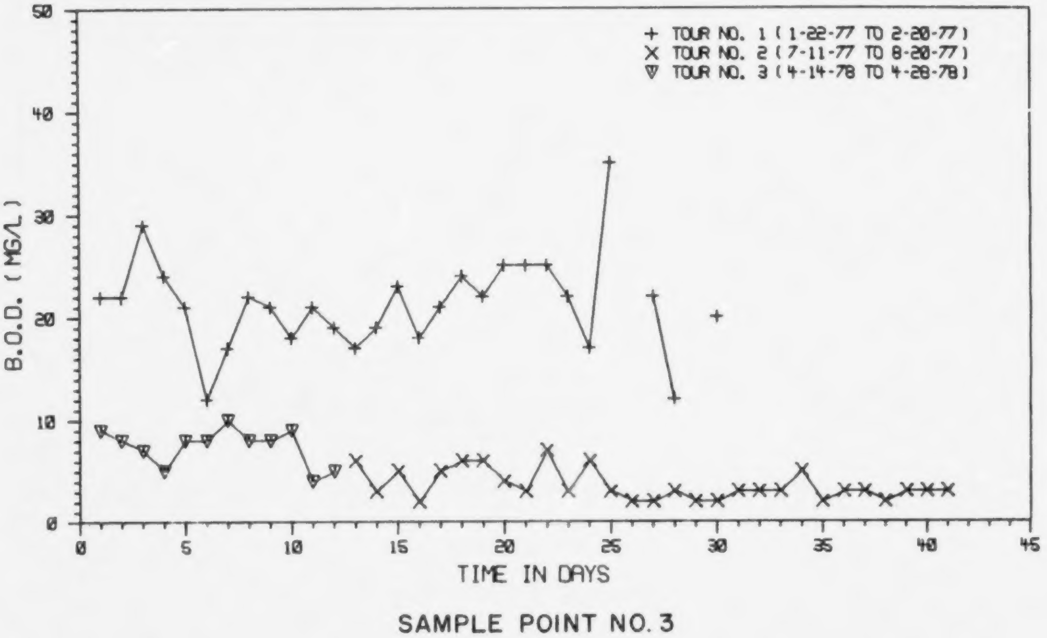


Figure 13. Final effluent BOD_5 concentration at Mt. Shasta, California.

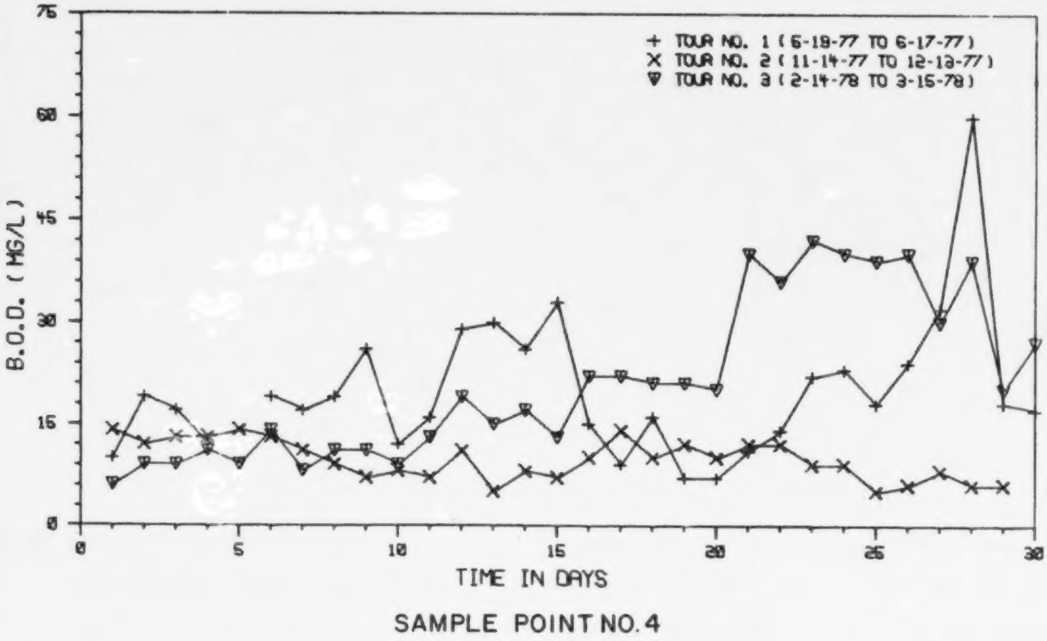


Figure 14. Final effluent BOD_5 concentration at Moriarty, New Mexico.

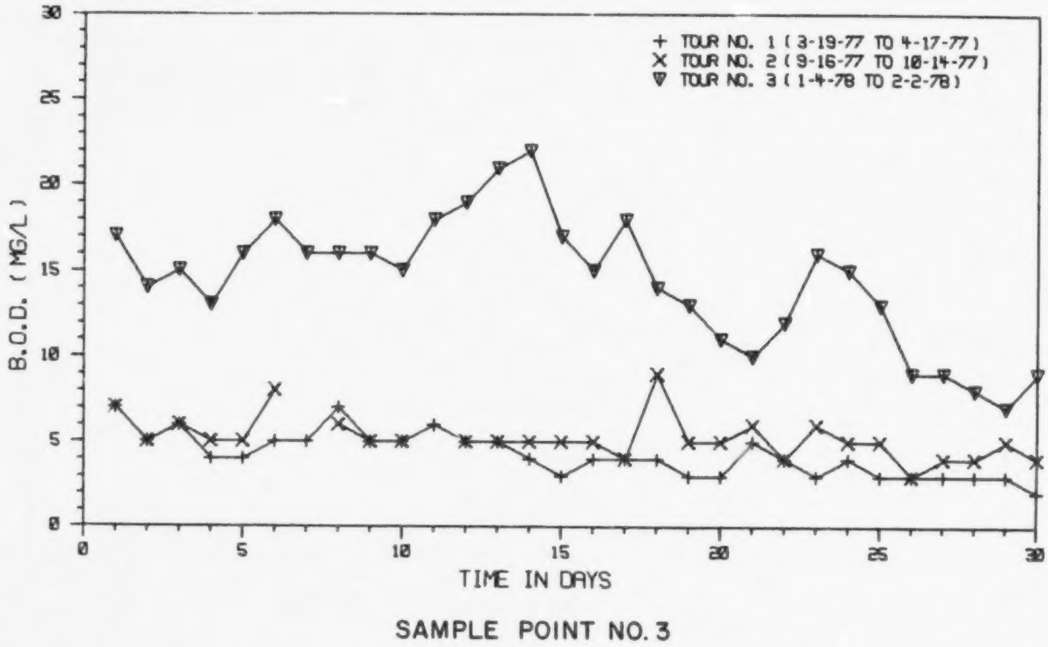
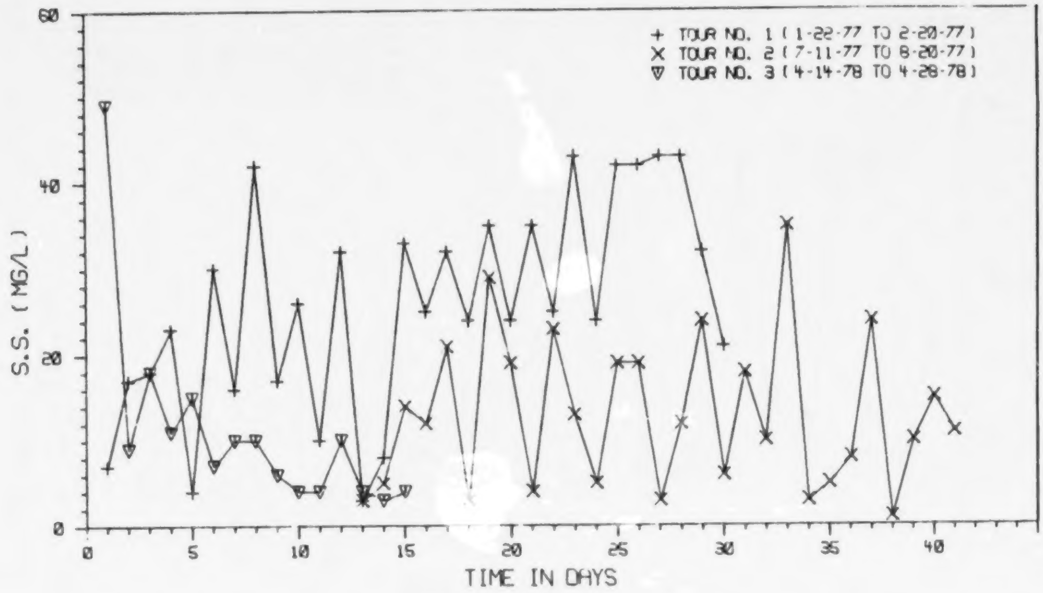
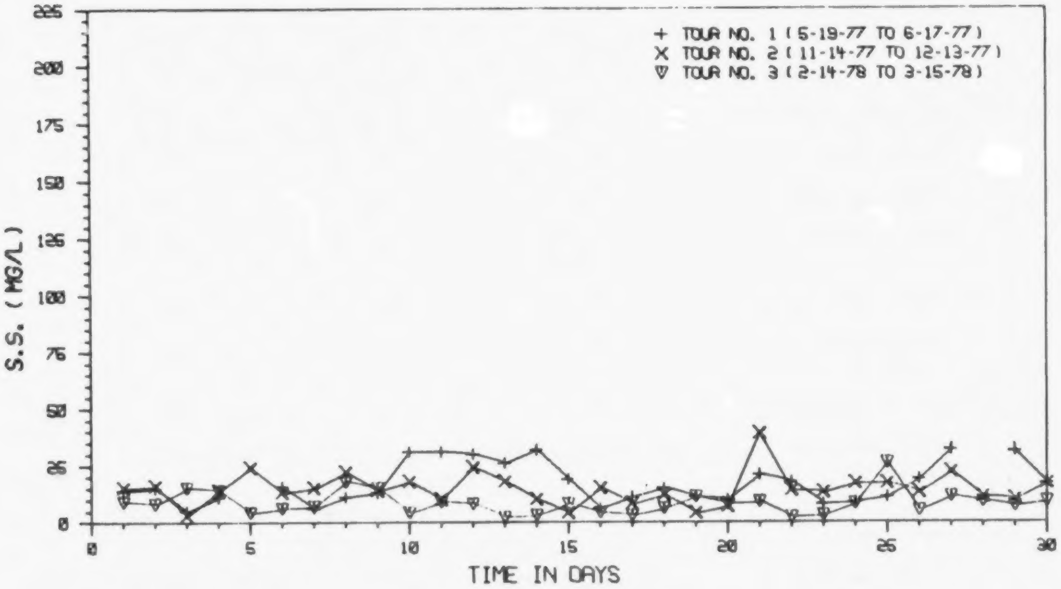


Figure 15. Final effluent BOD_5 concentration at Ailey, Georgia.



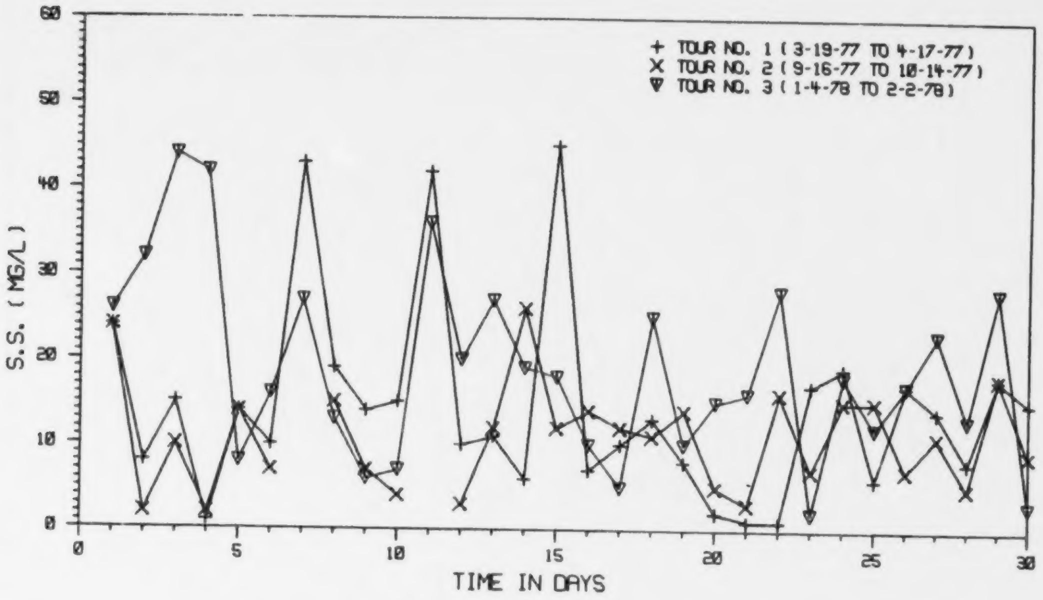
SAMPLE POINT NO. 3

Figure 16. Final effluent suspended solids concentration at Mt. Shasta, California.



SAMPLE POINT NO.4

Figure 17. Final effluent suspended solids concentration at Moriarty, New Mexico.



SAMPLE POINT NO.3

Figure 18. Final effluent suspended solids concentration at Ailey, Georgia.

Table 7. Summary of reported maintenance.

Job Description	Mount Shasta WPCF	Moriarty WWTF	Ailey STP
Daily operation and maintenance (daily monitoring)	(1.0 hr) x 7 days x 52 wks = 365	(1.0 hr) x 7 days x 52 wks = 365	(0.5 hr) x 5 days x 52 wks = 130
Filter cleaning	54*	28*	None
Filter raking	12 raking 16 mixing	13	22
Filter weed control	N.A.	None	26
Miscellaneous maintenance	N.A.	11	None
Grounds maintenance	42	8	28
Total reported man-hours	489 plus man-hours	425 man-hours	206 man-hours
Computed manpower requirements	2.4 man-year**	1 man-year**	1 man-year**
Actual reported manpower input	2.0 man-years***	0.28 man-year**	0.14 man-year**

* Man-hours with mechanical assistance
** Assuming 1500 man-hours = 1 man-year
*** Considering extra assistance for filter cleaning and weekend monitoring

SUMMARY

Experimental research and practical operation of full scale facilities has demonstrated the effectiveness of intermittent sand filtration for upgrading lagoon effluents. As with all wastewater treatment systems, performance is limited by proper operation and maintenance. However, less operator skill and manpower is required for operation of intermittent sand filters than with most conventional systems. Experience indicates that a high quality effluent may be achieved at a relatively low cost.

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LAND APPLICATION OF LAGOON EFFLUENTS

A. T. Wallace*

INTRODUCTION

Beginning in the 1950s, many small (and some not-so-small) communities adopted wastewater lagoons in one form or another in an attempt to meet treatment requirements with minimum investments in both capital construction and subsequent operation and maintenance expense. Many of these same communities, presented with state and federal directives to improve the quality of their lagoon effluents in the 1970s, have turned to some form of land application to avoid the obvious problems associated with discharge of nutrient and algae laden effluents to small receiving streams. Many other communities, forced to abandon individual, on-site disposal or seeking less costly, less complex treatment technology, have adopted combinations of lagoons and land application of effluent. In Idaho, for example, at least 15 federally funded projects involving the lagoon-land application concept have been approved and/or constructed since 1972. These range in scale from about 20,000 gallons per day to 7 million gallons per day. In addition, a few non-federally funded projects of a similar nature have been designed and built with state approval. Based upon recent (June 1978) telephone conversations with representatives of design firms and regulatory personnel, the situation is much the same in the states bordering Idaho.

It should come as no surprise that the reasons which lead to the selection of lagoons over more expensive and complex mechanical (either biological or P/C) plants should be the same as those for selecting land application over alternate effluent upgrading technologies in an over-whelming number of cases. Is it not fatuous to select lagoons originally because of their simplicity, and then proceed to upgrade them by tacking on a complete water filtration plant?

LITERATURE REVIEW

A review of the literature reveals that there are a great number of lagoon-land disposal systems throughout the U.S.

In EPA's "Survey of Facilities Using Land Application of Wastewater" a mail survey was conducted throughout the U.S. to gather information about communities using land application as a means of treatment or effluent disposal. Of 67 communities which responded, acknowledging their use of land application, 27 (40%) were employing lagoons as the method of preapplication treatment, 37 communities (including some of those using lagoons) reported the use of wastewater "holding ponds" varying in size from (V/Q) 5 days to over 30 days. One would certainly expect that effluent from such "holding ponds" would correspond closely in character to that from ponds designed specifically as treatment lagoons.

Some of these systems dated back to the early 1900s (one to 1889) and have been in continuous use. Most of the systems were subsequently visited by one of four project field investigators and very few problems are mentioned in their written commentary. In a few cases, the only problem mentioned was lack of suitable land to expand the system. As most of the systems reported on were in arid western states (California, Oregon, Washington, Arizona, and Texas accounted for most of them) it is not hard to understand why trouble-free operation prevails. In most instances the water would be of considerable value to the operator of the land disposal portion of the system. Thus, he could be expected to give immediate attention to any problem areas, such as odors or insects, so as not to jeopardize his rights to use the effluent.

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Unfortunately, reasonably good data on design and operation, especially with regard to process performance, are quite scarce. As many of the systems are very small and probably never monitored, this is not surprising. All but a very few of the land application portions of these systems are operated at low hydraulic rates, with application only to vegetated areas during the growing season. For these systems at least, adverse impacts are minimal and the owners have been largely relieved of the responsibility for extensive monitoring.

It is of interest to note that for low rate systems utilized only during the growing season with harvest of the crop practiced, that no definitive record of adverse impact to either soil or groundwater could be found in the literature search. Steel and Berg (1954) for example, concluded that the relatively minor changes produced in soils irrigated with effluents were neither especially beneficial nor injurious to the soil. Slight increases in organic matter were to be expected, together with increases in pore space and an improvement in crumb structure. These changes, which would be considered improvements, are off-set by a slight build-up in the salt content of the soil solution. It can be presumed that proper management techniques will allow indefinite use of low rate wastewater irrigation systems which apply lagoon effluents of reasonably good quality.

Malhotra and Myers (1975) reported some data on groundwater quality beneath and adjacent to the land disposal area of two combination lagoon-land application areas. Increases above background were observed for chloride, nitrate, specific conductance, and alkalinity. However, the increases were tolerable relative to water quality criteria for most purposes. Increases were higher for higher hydraulic loadings and also for sandier soils.

Hicken et al. (1978) performed a 2-year field study using three different application rates (2, 4, and 6 inches per week) on four different soil types, ranging from a clay to a sandy loam texture. Increases in specific conductance and sodium in the percolate were high enough to prevent additional reuse of the drainage water for irrigation. Increases in nitrate and organic carbon were also observed, however, it was concluded that these increases were primarily caused by leaching of nitrates and carbon which were present in the soil prior to the initiation of irrigation with lagoon effluent.

Relative to high rate (for example, rapid infiltration) systems, the situation is quite different. These are still treatment, as opposed to disposal, systems although they have been referred to otherwise by some engineers. However, because of the heavier hydraulic (concomitantly all constituent) loadings, they have a greater impact on both soil and groundwater. Bouwer (1978) has suggested that such systems only be allowed when groundwater quality is either of no consequence or when the percolate can be controlled (as by stream recharge or pumping). Their use has been discouraged in some states, for example Michigan (Malhotra and Myers, 1975).

An interesting observation was made by Bouwer relative to the effect of algae in the clogging process in rapid infiltration systems. The data accumulated so far at Flushing Meadows (Phoenix) seem to indicate that algae have a greater clogging potential than an equal mass of suspended solids from secondary treatment (activated sludge) at the high loading rates employed in that system (over 45 inches/week). This fact is ordinarily of no consequence for the more common low-rate systems (around 1 to 3 inches/week). However, Hicken et al. (1978) observed prolific algae growths on nonvegetated control plots at application rates of 2 to 6 inches per week. Contrary to Bouwer's observations, they concluded that the algae mats did not increase the hydraulic impedance on these sites. Perhaps the explanation lies in the great difference in application rate between the two sets of observations.

A REVIEW OF TWO INTERESTING PROJECTS

Davis, California

The background work leading to the upgrading of the Davis, California, lagoon system is described by Tucker et al. (1977). The present system consists of a complete battery of preliminary and primary treatment units. These include coarse screening, prechlorination, comminution, pre-aeration/ grit removal, and primary sedimentation. These units are credited with 35 percent removal of BOD and 65 percent removal of suspended solids. The next stage of treatment consists of three facultative ponds operated in parallel. The detention time is 39 days based on an inflow of 5 mgd. Recirculation is provided via two high flow (15 mgd), low head pumps and recirculation channels. A recirculation ratio of 0, 3, or 6 can be provided.

Through 1975, the city met all Regional Water Quality Board discharge requirements except that for suspended solids. The target level was 30 mg/l and their 1975 average was 74 mg/l with the highest monthly average recorded as 93 mg/l. Interestingly, the lagoon flow scheme was modified from parallel to series operation for a few months to try to reduce effluent suspended solids with no apparent success.

Alternatives which received consideration included replacement of the existing lagoons as well as lagoon effluent treatment. Table 1 summarizes the costs of the three lagoon effluent upgrading options for Davis.

On an annual cost basis, assuming a 20 year life for physical improvements and a 7 percent interest rate, the overland flow scheme shows the least cost. Although only the assumed cost of the land ($360,000) was included in the salvage value of the project, at least a portion of some of the improvements (terrace construction and distribution system) which might make the property more valuable to a subsequent agricultural operator might have been included as well. In addition, no income from the harvest of grass hay was included. Both of these factors would tend to make the overland flow option even more attractive.

In terms of environmental impact, a positive benefit was assumed due to the increase in wildlife habitat. The negative aspects of enhanced insect breeding would have to be solved by careful design and management.

A 5-month pilot scale program was undertaken on three 50 ft by 100 ft plots seeded with annual rye grass. The slope of the plots was 2 percent. Data collection spanned the period from 11 November 1975 to 27 March 1976. From this program, the maximum practical hydraulic loading was identified as 1.2 inches per day. Beyond this loading, effluent (runoff) quality became marginally acceptable. As the grass was observed to grow continuously during this period, it was concluded that year-around operation may be feasible in this climatic area.

Table 1. Capital and O&M costs for lagoon effluent upgrading options for Davis, California. (September 1977 Construction Costs ENRCCI - 3200.)

Option	Capital Cost	Annual O&M
Coagulation-Flocculation-Sedimentation	$1.51 mill	$278,000
Intermittent Sand Filtration	$3.52 mill	$ 79,000
Overland Flow	$1.98 mill	$ 88,000

Muskegon County (Michigan)

No treatment of this subject would be complete without mention of this immense project. A complete description is available (Demirjian, 1975) and only the highlights are summarized in the following paragraphs.

The system is located on 11,000 acres of formerly unproductive sandy land. The total design capacity is 42 mgd of combined domestic-industrial waste. Biological treatment is provided by three aerated lagoons, operated in either series or parallel, with a detention period of 3 days. Two storage lagoons provide approximately 4 months of storage at the design flow.

Effluent may be taken from storage or directly from the aerated lagoons via a small settling pond, chlorinated and applied to the land. The irrigation system employs center pivot machines of special design and covers 6,000 acres. The water budget calls for an application of 3.8 inches per week (including precipitation) for an 8 month irrigation season. Field (feed) corn is the primary crop.

An important aspect of the Muskegon system is the control of subsurface water and percolate. An extensive drainage system, utilizing tiles, wells, and ditches, was required to control the groundwater table and prevent waterlogging in the aerobic (renovation) zone.

Because of the size of the project and some of its political ramifications, a rather large sum of money has been allocated for its study. Thus, in contrast to many of its lesser (in scope) counterparts, a great deal of performance data is being accumulated for the Muskegon system. Some of the most important findings to date are: 1) No significant accumulations of most heavy metals observed in either grain or plant tissue. Cd and Zn concentrations increased in the tissue but not in the grain. 2) The nitrogen content of the wastewater was insufficient to support optimum corn growth, thus had to be supplemented. Application of the optimum amount of nitrogen resulted in a decrease in nitrogen lost in the percolate. 3)

Excellent performance is being experienced relative to organic removals through the land application portion of the system. The same is true for nitrogen forms and phosphorus. 4) Bacterial quality of the percolate, however, has been erratic and often above the adopted levels (200 fecal coliform per 100 ml).

In summary, the Muskegon County system must be viewed as a very successful example of the technology which we are discussing. This in spite of severe problems encountered with the system hardware during start-up and a very shakey first season of operation. The system is a unique example of land application of lagoon effluent on a grand scale and for a few years had produced two polarized camps; one of ardent supporters and one of equally ardent opponents. The fact that one seldom hears vicious attacks on the system in public anymore is probably significant.

OBSERVED PLANNING AND DESIGN DEFICIENCIES

A total of 16 projects utilizing a combination of lagoons and land application were carefully reviewed in the preparation of this paper. Of these projects, 12 were located in Idaho, 2 in Wyoming, and 2 in Oregon. The preliminary plans (now called "facilities plans") for these projects cover a period from 1971 to 1978. Thus they should (and do) reflect both improvements in the state-of-art and tightening requirements by the state regulatory agencies and the EPA relative to land application. For example, plans prepared during the years 1971-1973 contain very little of the site-specific data (climate, geology, groundwater, and soils) which are routinely found in more recent reports. However, even among these later, more detailed, reports there is a wide divergence in the general approach. Much important information seems to be left out in some plans. Significant design parameters are developed by many, often widely different, techniques in others. Design parameters are often assumed without benefit of site work in some plans and these same parameters are derived from field measurements in others without specifying the methods used, tabulating the raw data, or showing the calculated statistics of the measurements. Incorrect calculations were observed in a few cases. In another few cases, seemingly representative and consistent data were obtained and properly analyzed, but subsequent design loadings were chosen which bore no apparent relationship to the derived quantity or quantities. The inconsistency in basic assumptions, particularly with regard to nitrogen balances, was the most puzzling observation. In addition, assumptions which were obviously derived from literature searches were neither consistent (from one firm to the next) nor properly referenced.

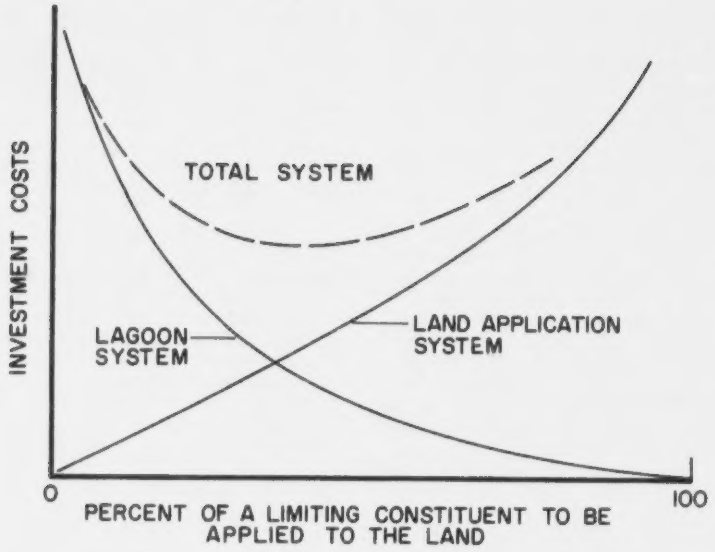
Many of these deficiencies could have been corrected by faithfully following a comprehensive set of guidelines, for example, ADDENDUM NO. 2 to Recommended Standards for Sewage Works (1968 Edition), Great Lakes - Upper Mississippi Board, "Ground Disposal of Wastewaters" available as of April 1971. A more recent, and much more detailed, guidance document has been made available through the EPA (Oct. 1977).

In summary, the person(s) preparing the plans reviewed, with a few notable exceptions, seemed to be quite comfortable with all aspects of the analysis except that dealing with the land application portion.

This observation extends to the important relationship between the lagoon effluent quality and the land application portion of the project. Lagoon sizing criteria were quite arbitrary and in no way related to the ability of a particular soil to accept effluent, or to a desired endpoint, for example, a given groundwater quality. The relationship depicted by Figure 1 was not considered, even in a gross way, during any of the planning efforts in hopes of identifying a truly least-cost system. However, in fairness to the engineering firms, it should be pointed out that state regulations, at least in Idaho and Oregon, force the system configuration toward the left side of the graph, thus ruling out any possibility of achieving a cost-effective solution.

At present, only 26 states have regulations (or guidelines) pertaining to land application of wastewaters. Of these, 21 require secondary treatment as the minimum level of pre-treatment. It is plain that where direct discharge of effluents would be allowed, land application will never be part of a cost-effective solution for these 21 states.

There are a lot of unanswered questions relative to the fate of specific constituents in the soil and groundwater (not to mention aerosols from spray operations). Certainly we need to make efforts to fill these gaps in our knowledge. (See, for example references 10 and 11.) However, many of



(AFTER : M.R. OVERCASH, ET AL, INDUSTRIAL WASTE LAND APPLICATION, AICHE TODAY SERIES 1976)

the same questions exist relative to the fate of pollutants discharged to surface waters. We can't afford to cease either method of disposal while we wait for every last question to receive a satisfactory answer. There are some (Bernarde, 1973) who are of the opinion that land disposal is potentially far less hazardous, from the standpoint of communicable disease transmission, than disposal into rivers and streams.

As in many fields of technology, we are forced to "refine as we go"; making some mistakes and learning from them, then incorporating what we have learned into the next system. In the meanwhile, we must change the way in which we think about some of the impacts. Phelps, in 1944, decried the enactment of laws "calling for a given standard of treatment regardless of the character or use of the stream." He espoused tailoring design of treatment works to protect all beneficial uses of the receiving stream while utilizing the assimilative capacity to the maximum extent consistent with the preservation objective. I believe that Phelps was right in 1944 and he is still right. Also, his philosophy applies equally well to land application systems. Regulatory agencies have recognized the necessity of establishing a "mixing zone" next to outfalls, within which water quality standards do not apply as strictly as they do beyond the mixing zone. Why then do we have so much concern for groundwater quality directly below the land application site? Emrich (1978) has suggested general adoption of a concept analogous to the "mixing zone" for evaluating land application systems and a similar concept has already been applied (Wallace, 1973).

SUMMARY REMARKS

When considering land application as a technique for upgrading lagoon effluents, there is no reason to allow your thinking to be too narrowly constrained. For example, some states do not allow land application outside of the period which normally constitutes the "growing season." If strict control of nitrogen in the groundwater has been identified as a critical system requirement, there is good reason for limiting the period of irrigation. However, when nitrogen considerations are not important, why limit the period for application? Surely BOD, suspended solids, phosphorus and many other constituents will be transformed or removed during the cold weather if there are no physical limitations to applying wastewater. Although ice formation has restricted application for some systems (Armstrong, Borrelli and Burman, 1978), by careful design and management others have operated successfully at temperatures down to -35°F (Anderson, 1975).

The spray system at Thayne, Wyoming, discussed by Armstrong et al. was modified to employ rapid infiltration during the coldest part of the winter to avoid the problems caused by ice formation around the sprinkler heads (Orton, 1978). This technique would prove useful at many locations to avoid the high cost of long-term effluent storage. In addition, rapid infiltration or flood irrigation might easily be used in a portion of the buffer areas prescribed by regulatory agencies around spray fields to promote more efficient use of the land.

Other possibilities for the use of innovative technology would involve the use of artificial marshes, after the lagoon system, but prior to land application. Such a scheme might be used when increased nutrient or algae removal became necessary.

It seems certain that a great number of lagoon systems will be upgraded through the use of land application of effluents and many lagoon-land application systems will be designed and built as integrated units for the purposes of fostering nutrient recycle. The land application portion of these systems will ordinarily be considered as innovative or alternative technology, thereby qualifying these subsystems for an additional 10 percent federal funding. In response to this, design firms will gradually develop the expertise necessary to plan and design these systems for maximum public benefit and minimum adverse impact.

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DISINFECTION OF LAGOON EFFLUENTS

Bruce A. Johnson*

For many years, wastewater stabilization ponds have been used successfully in all parts of the United States to provide treatment of domestic and industrial wastes. Since ponds require very little operator control and maintenance for successful performance, they have been particularly popular among small and rural communities, where land is relatively inexpensive. However, as the result of more stringent state and federal discharge standards, many pond systems are unable to meet new requirements. This is particularly true with respect to bacterial reduction. Therefore, disinfection must be considered as a means of upgrading pond effluent to meet bacteriological discharge standards.

LITERATURE REVIEW--CHLORINATION OF POND EFFLUENT

Since chlorine, at present, is less expensive and offers more flexibility than other means of disinfection, chlorination, is the most practical method of reducing bacterial populations. However, there is evidence that chlorination of wastewater high in organic nitrogen content, such as stabilization pond effluent, may be accompanied by adverse effects. Therefore, several major research projects have been undertaken in recent years to investigate the chlorination of pond effluents.

White (1973) has suggested that chlorine demand is increased by high concentrations of algae commonly found in pond effluents. It was found that to satisfy chlorine demand and to produce enough residual to effectively disinfect algae laden wastewater within 30-45 minutes, a chlorine dose of 20-30 mg/l was required. Kott (1971) also reported increases in chlorine demand as a result of algae, but found that a chlorine dose of 8 mg/l was sufficient to produce adequate disinfection within 30 minutes and that if contact times are kept relatively short, no serious chlorine demand by algae cells is encountered. Of course, the amount of chlorine demand exerted is highly variable. Dinges and Rust (1969) found that for pond effluents, a chlorine demand of only 2.65 to 3.0 mg/l was exerted after 20 minutes of contact. Brinkhead and O'Brien (1973) found that at low doses of chlorine, very little increase in chlorine demand is attributable to algae, but at higher doses, the destruction of algae cells greatly increases demand. This is because dissolved organic compounds released from destroyed algae cells, as explained by Echelberger et al. (1971), are oxidized by chlorine and thus increase chlorine demand.

Another concern regarding the chlorination of pond effluents is the effects on biochemical oxygen demand (BOD_5) and chemical oxygen demand (COD) Brinkhead and O'Brien (1973) and Echelberger et al. (1971) found that for higher chlorine doses, increases in BOD_5 due to destruction of algae cells were observed. Echelberger et al. (1971) also reported increases in soluble COD. Hom (1972) found that when 2.0 mg/l chlorine was applied to pond effluent, the BOD_5 measured was 20 mg/l. However, when 64 mg/l chlorine was applied, the BOD_5 increased to 129 mg/l. However, Zaloum and Murphy (1974) observed a 40 percent reduction of BOD_5 resulting from chlorination. Dinges and Rust (1969) also reported reductions of BOD_5. Kott (1971) has suggested that increases in BOD_5 may be controlled by using fairly low chlorine doses coupled with relatively long contact periods.

The formation of toxic chloramines is also of concern in chlorinating pond effluents. These compounds are found in waters high in ammonia concentration and are extremely toxic to aquatic life found in receiving water. For example, Zillich (1972) has determined that a chloramine concentration of 0.06 mg/l is lethal to trout.

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Not all of the side effects of chlorinating pond effluents are detrimental. Kott (1973) observed reductions of suspended solids (SS) as a result of chlorination. Dinges and Rust (1969) reported reductions of volatile suspended solids (VSS) by as much as 52.3 percent and improved water clarity (turbidity) by 31.8 percent following chlorination. Echelberger et al. (1971) reported that chlorine enhances the flocculation of algae masses by causing algae cells to clump together.

Additional research has been and is being conducted regarding improvement of chlorination efficiency and chlorine contact tank design for wastewater treatment in general. Additional research, however, directed specifically to chlorination of pond effluents, is needed to more fully understand control and design parameters for this application of chlorination.

BASIC PRINCIPLES OF CHLORINATION

To understand the effects of chlorinating stabilization pond effluents, it is necessary to review the basic principles of chlorination. When chlorine gas is used the gas reacts with water to form hypochlorous acid (HOCl). In a pure water system, the reaction is as follows:

$$Cl_2 + H_2O \longrightarrow HOCl + H^+ + Cl \;.\;.\; (1)$$

The hypochlorous acid then dissassociates to form OCl^- and H^+

$$HOCl \rightleftharpoons H^+ + OCl^- \;.\;.\;.\;.\;.\;.\;.\; (2)$$

When $Ca(OCl)_2$, for example, is used to chlorinate, OCl^- is formed by the following reaction:

$$Ca(OCl)_2 \longrightarrow Ca^{++} + 2OCl^- \;.\;.\;.\;.\; (3)$$

The OCl^- is then free to form hypochlorous acid in contact with hydrogen ions. Chlorine in the form of HOCl or OCL^- is referred to as free chlorine. Both forms of free chlorine are powerful disinfectants and react quickly to destroy bacteria and most viruses.

In wastewater, such as stabilization pond effluents, various chemical components react with free chlorine to form compounds which are ineffective as disinfectants. That is, the rates of reactions between chlorine and these components are faster than the rate at which chlorine attacks and kills bacteria and viruses. Fe^{++}, Mn^{++}, $NO_2^=$ and $S^=$ are common reducing agents which combine readily with chlorine to prevent it from disinfecting. A typical reaction is as follows:

$$H_2S + 4Cl_2 + 4H_2O \longrightarrow H_2SO_4 + 8HCl \;.\;.\;.\; (4)$$

Free chlorine also reacts with ammonia found in wastewater to form a series of compounds known as chloramines. Although chloramines are less than 5 percent as efficient as free chlorine in destroying bacteria and viruses, they do play an important role in disinfection because they are fairly stable and can continue to provide disinfection for some time after application. The common forms of chloramines, or combined chlorine, as they are referred to, are monochloramine, dichloramine, and nitrogen trichloride. The reactions for their formation are as follows:

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O \;.\;.\;.\;.\;.\; (5)$$

$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O \;.\;.\;.\;.\; (6)$$

$$NHCl_2 + HOCl \rightleftharpoons HCl_3 + H_2O \;.\;.\;.\;.\;.\; (7)$$

In some cases, chlorination is used as a treatment step to drive off undesirable ammonia. This is known as breakpoint chlorination. Basically, chlorine is added until all the chlorine has reacted to form chloramines. With the addition of more chlorine, the ammonia is converted to nitrogen gas and driven off. Any additional chlorine added beyond that point is maintained in solution as free chlorine residual. The mechanisms involved are fairly complex, but the overall reaction may be represented as follows:

$$2NH_3 + 3HOCl \longrightarrow N_2\uparrow + 3HCl + 3H_2O \;.\;.\; (8)$$

A comparison of ideal breakpoint chlorination versus wastewater breakpoint chlorination is represented in Figure 1. Because the chlorine dose necessary to reach the breakpoint in wastewater is much higher than the dose necessary to achieve adequate disinfection, breakpoint chlorination is seldom used in the treatment of wastewater.

FIELD STUDY--CHLORINATION OF POND EFFLUENT

To add to the knowledge concerning the chlorination of pond effluents, a study was conducted during 1975-76 at Utah State University to evaluate pond chlorination practices and facilities under varying seasonal conditions. The experimental chlorination facilities were constructed with the capabilities of treating either primary or secondary pond effluent. Four systems of identically designed chlorine mixing and contact tanks, each capable of treating 50,000 gallons per day, were constructed. Three of the four chlorination systems were used for directly treating pond effluent. The effluent treated in the fourth system was filtered through an intermittent sand filter prior to chlorination to remove algae. The filtered effluent was also used as the solution water for all four chlorination systems. An overall schematic of the chlorination system is illustrated in Figure 2.

Following recommendations by Collins, Selleck, and White (1971), Kothandaraman and Evans (1972 and 1974), and Marske and Boyle (1973), the chlorination systems were constructed to provide rapid initial mixing followed by chlorine contact in plug flow reactors. A serpentine flow configuration having a length to width ratio of 25:1, coupled with inlet and outlet baffles, was used to enhance plug flow hydraulic performance. The chlorine mixing and contact tanks are illustrated in Figure 3. The maximum theoretical detention time for each tank was 60 minutes, while the maximum actual detention time averaged about 50 minutes.

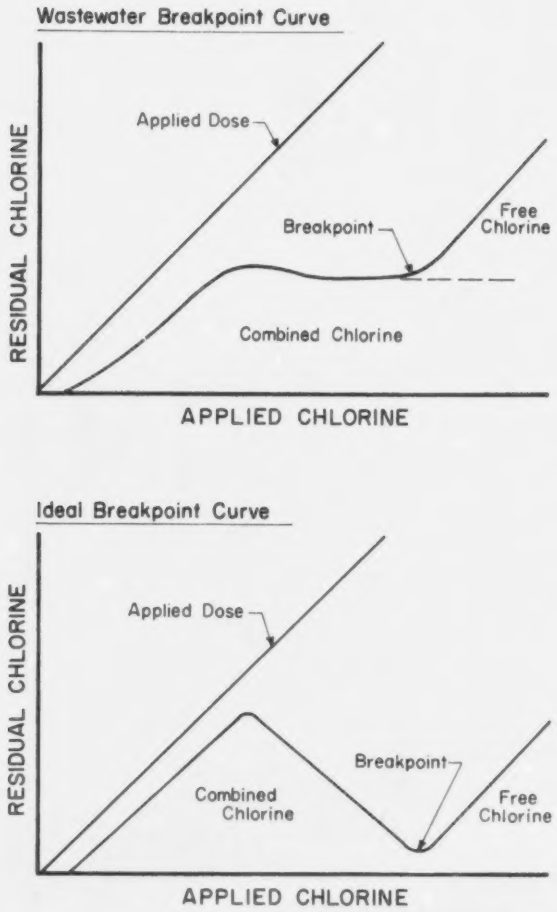


Figure 1. Comparison between ideal and wastewater chlorination curves.

The pond effluent was chlorinated at doses ranging from 0.25 to 30.0 mg/l under a variety of contact times, temperatures, and seasonal effluent conditions from August 1975 to August 1976. A variety of chemical, physical, and bacteriological parameters were monitored during this period in evaluating the chlorination of pond effluents. A series of laboratory experiments as also conducted to compliment the field study. Some of the major findings of this study are summarized as follows:

1. Sulfide, produced as a result of anaerobic conditions in the ponds during winter months when the ponds are frozen over, exerts a significant chlorine demand. This is illustrated in Figure 4. For sulfide concentrations of 1.0 - 1.8 mg/l, a chlorine dose of 6 to 7 mg/l was required to produce the same residual as a chlorine dose of about 1 mg/l for conditions of no sulfide.

2. For all concentrations of ammonia encountered, it was found that adequate disinfection could be obtained with combined chlorine residual in 50 minutes or less of contact time. Therefore, breakpoint chlorination, and the subsequent production of free chlorine residual, was found to be rarely, if ever, necessary in disinfecting pond effluent.

3. In considering the chlorination of pond effluents, concern has been expressed that the lysis of algae cells would cause an increase in chemical oxygen demand (COD). It was found that total COD is virtually unaffected by chlorination. Soluble COD, however, was found to increase with increasing concentrations of free chlorine only. This increase was attributed to the oxidation of suspended solids by free chlorine. Increases in soluble COD versus free chlorine residual are illustrated in Figure 5.

4. Some reduction of suspended solids, due to the break down and oxidation of suspended particulates, and resulting increases in turbidity were

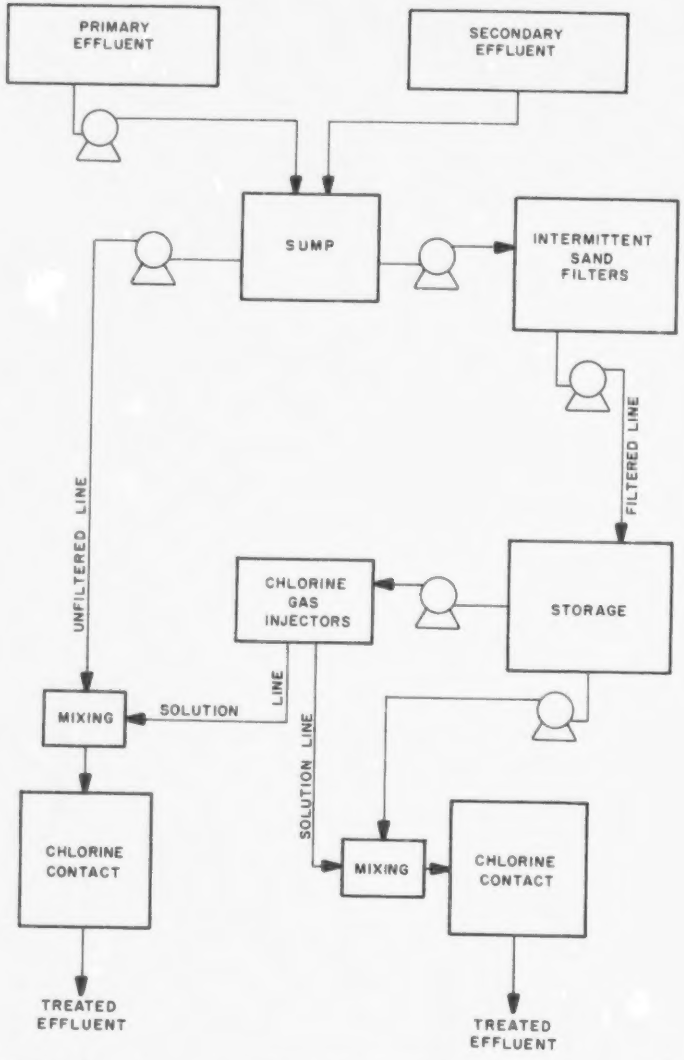

PRIMARY EFFLUENT
SECONDARY EFFLUENT
SUMP
INTERMITTENT SAND FILTERS
FILTERED LINE
UNFILTERED LINE
STORAGE
CHLORINE GAS INJECTORS
SOLUTION LINE
SOLUTION LINE
MIXING
MIXING
CHLORINE CONTACT
CHLORINE CONTACT
TREATED EFFLUENT
TREATED EFFLUENT

Figure 2. Experimental chlorination schematic.

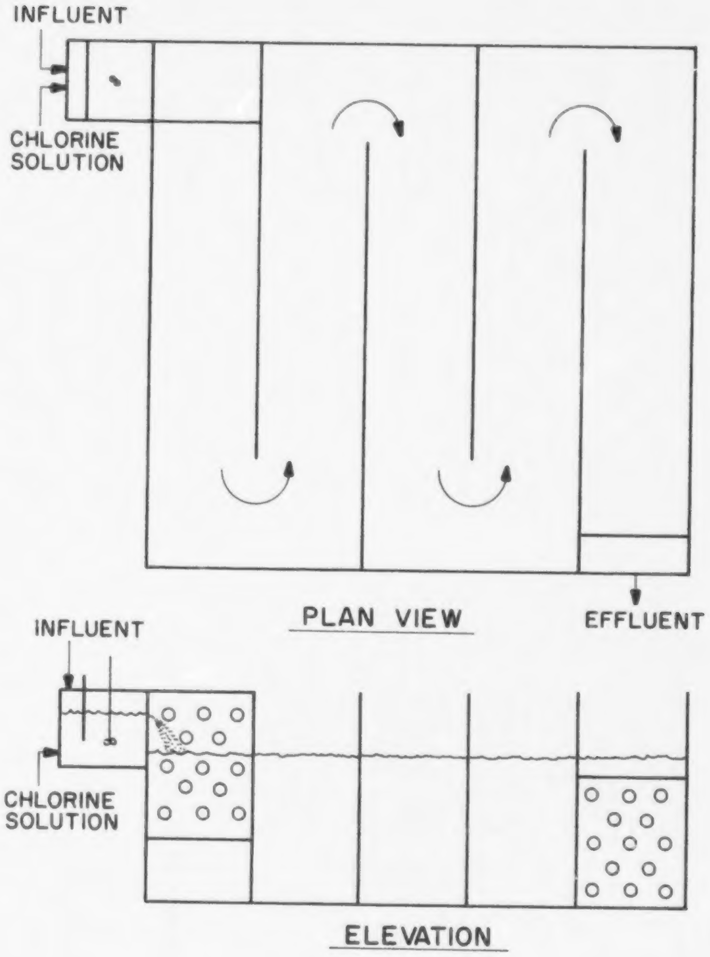

INFLUENT
CHLORINE SOLUTION
PLAN VIEW
EFFLUENT
INFLUENT
CHLORINE SOLUTION
ELEVATION

Figure 3. Chlorine mixing and contact tanks.

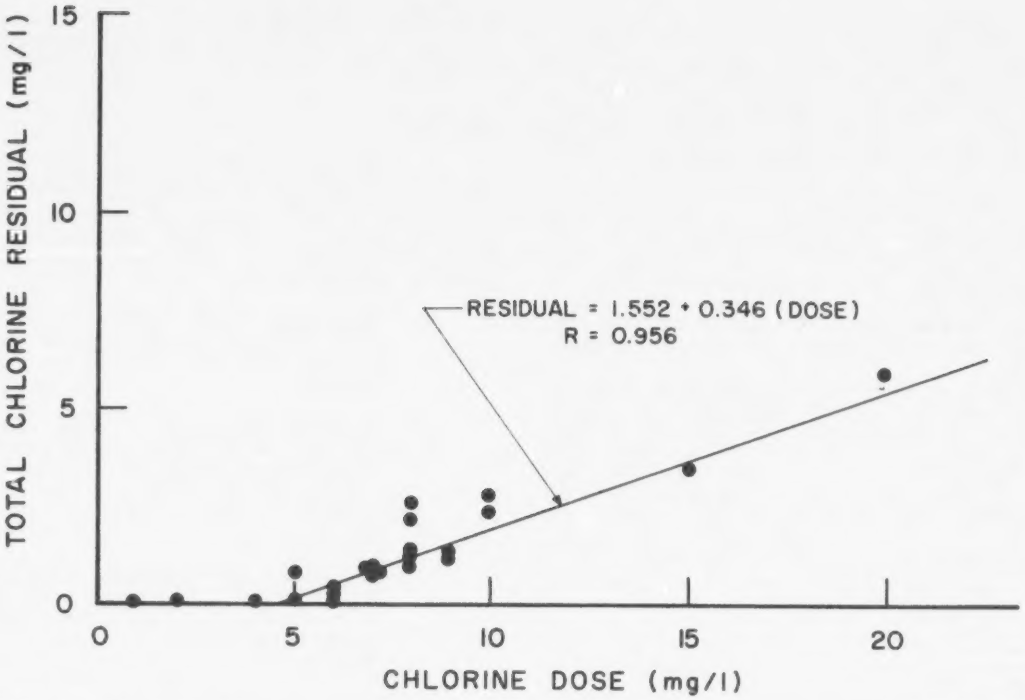


Figure 4. Chlorine dose vs. residual for initial sulfide concentrations of 1.0 - 1.8 mg/l.

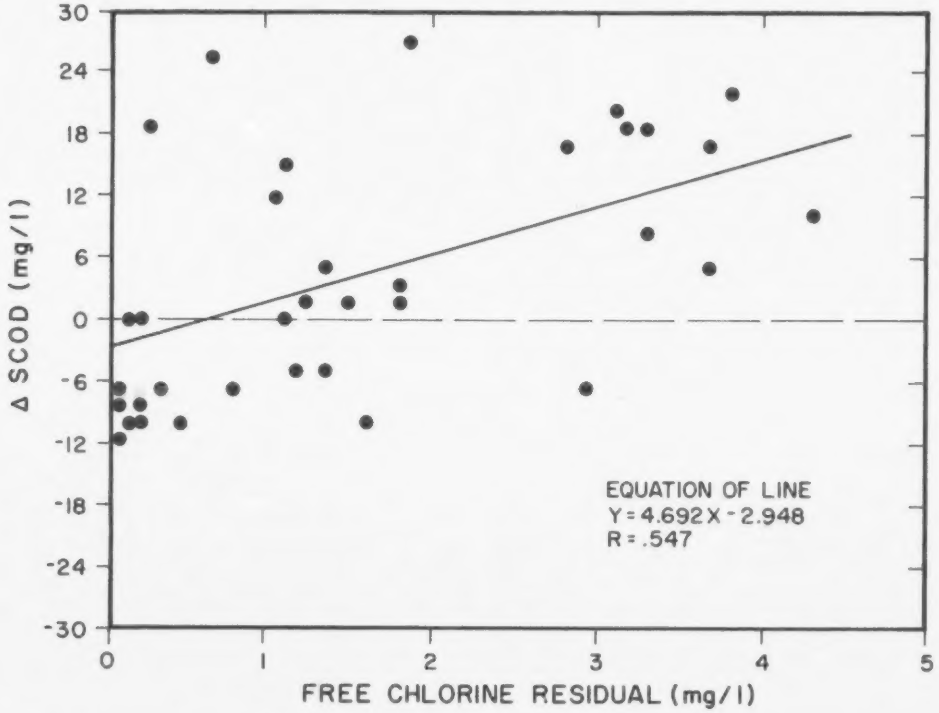


Figure 5. Changes in soluble COD vs. free chlorine residual--unfiltered lagoon effluent.

attributed to chlorination. However, this reduction was found to be of limited importance in comparison with reductions of suspended solids resulting from settling. Suspended solids were found to be reduced by 10-50 percent from settling in the contact tanks.

5. Filtered pond effluent was found to exert a lower chlorine demand than unfiltered pond effluent, due to the removal of algae. This is comparatively illustrated in Figure 6. The rate of exertion of chlorine demand was determined to be directly related to chlorine dose and total chemical oxygen demand.

6. A summary of coliform removal efficiencies as a function of total chlorine residual for filtered and unfiltered effluent is illustrated in Figure 7. It was found that the rate of disinfection is a function of the chlorine dose and bacterial concentration. Generally, the chlorine demand was found to be about 50 percent of the applied chlorine dose except during periods of sulfide production.

7. Disinfection efficiency was found to be highly temperature dependent. At colder temperatures, the reduction in the rate of disinfection was partially offset by reductions in the exertion of chlorine demand. However, the net effect was a reduction in the chlorine residual necessary to achieve adequate disinfection with increasing temperature for a specific contact period.

8. In almost all cases, adequate disinfection was obtained with combined chlorine residuals of between 0.5 and 1.0 mg/l after a contact period of approximately 50 minutes. This indicated that disinfection can be achieved without discharging excessive concentrations of toxic chlorine residuals into receiving waters. Also, it was found that adequate bacterial removal can be achieved with relatively low doses of applied chlorine during most of the year.

DESIGN CRITERIA

The objective of chlorinating pond effluents is to reduce bacterial populations to acceptable levels while minimizing the applied chlorine dose required and any adverse effects which might be imparted to receiving waters. Typically, fecal coliform bacteria must be reduced to less than 200/100 ml on a daily average. To accomplish this objective, careful attention must be given to design and operation of chlorination facilities, particularly with respect to mixing and hydraulic performance of the contact chambers. A summary of some major design criteria is contained in the following sections.

Mixing

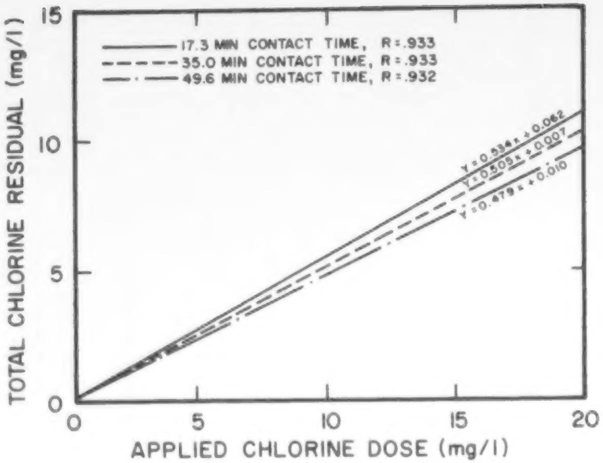
Rapid, initial mixing is probably the most important aspect of efficient disinfection and should be accomplished within about 5 seconds and prior to entering a contact chamber. Several methods of mixing are:

1. Hydraulic jump. This is probably the best method for obtaining rapid mixing in an open channel.
2. Mechanical mixers. The mixer should be located immediately downstream from the point of chlorine injection and the mixing chamber should be as small as possible.
3. Turbulent flow through restricted reactor. This is a highly effective method in most cases, particularly in a closed conduit.
4. Injections of chlorine into full flowing pipe. Probably the least efficient method and should not be used in pipes 30 inches in diameter and larger.

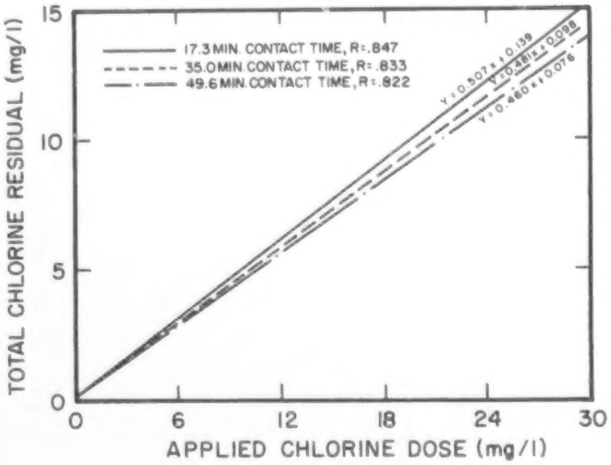
Contact Chambers

Chlorine contact chambers should be designed to provide at least 1 hour detention time at average flow and 30 minutes detention time at peak hourly flow, whichever is greater.

1. Hydraulic performances. Ideally, contact tanks should be designed to produce near ideal plugflow. Hydraulic conditions may be considered acceptable when the modal value, as determined by dye tracer tests, is greater than 0.6. (The modal value is determined by dividing the time which corresponds to the highest point of the tracer residence time distribution curve by the theoretical detention time.) A contact tank constructed with baffles parallel to the longitudinal axis of the chamber or a long narrow channel having a minimum length to width ratio of 40:1 is recommended. Cross-baffles in shallow chambers may also be necessary to reduce short circuiting caused by wind currents.

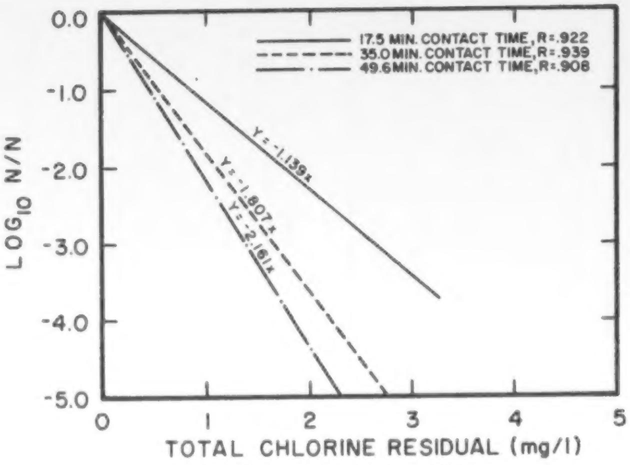


(a) Filtered Effluent

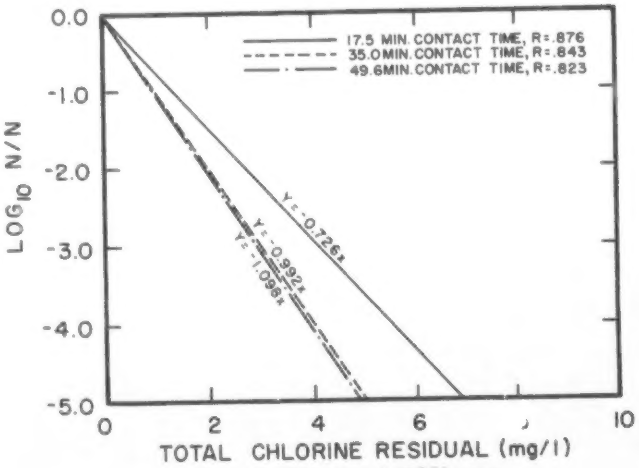


(b) Unfiltered Effluent

Figure 6. Chlorine dose vs. total residual--filtered and unfiltered effluent.



(a) Filtered Effluent



(b) Unfiltered Effluent

Figure 7. Coliform removal efficiencies--filtered and unfiltered effluent.

2. Solids removal. Baffles should be provided for removal of floating solids while provisions should be made for draining the contact tank and removing settled solids. Duplicate contact chambers should be provided so that one can remain in service while the other one is being cleaned. Channel widths should be wide enough to allow easy access for cleaning.

3. Outfall lines. Outfall lines may be used for chlorine contact if they are flowing full at all times, exhibit the proper detention time, and preclude infiltration and exfiltration.

Dechlorination

Dechlorination facilities may be required in a case-by-case basis where chlorinated effluent is being discharged to biologically sensitive receiving waters and/or to meet established chlorine receiving water quality criteria.

Chlorine Supply

Sufficient storage should be provided to provide at least one spare cylinder for each one in service. The chlorination room should be maintained at at least 55°F, but heat or direct sunlight must never be applied directly to the cylinders. The maximum withdrawal rate for 100 and 150 pound cylinders should be limited to 40 lbs per day and, for 2000 pound cylinders, to 400 lbs per day per cylinder. Scales and cylinder handling equipment should be provided. An automatic switch-over system should be installed at facilities having less than continuous operator attendance.

Piping and Valves

All piping and valves should be approved by the Chlorine Institute. Recommended piping is as follows:

1. Supply piping between cylinder and chlorinator should be Sc. 80 black seamless steel pipe with 2000 pound forged steel fitting. Unions should be ammonia type with lead gaskets.

2. Chlorine solution lines should be Sc. 80 PVC, rubber-lined steel, saran-lined steel, or fiber cast pipe approved for moist chlorine use. Valves should be PVC, rubber-lined, or PVC lined.

3. Injector line between chlorinator and injector should be Sc. 80 PVC or fiber cast approved for moist chlorine use.

Chlorinators

Chlorinators should be sized so that they are capable of providing a dose of at least 10 mg/l in the effluent. The maximum feed rate required may vary from pond to pond and should be determined on an individual basis. Basically, there are two types of chlorine gas chlorinators.

1. Direct feed. This type of chlorinator should be used only in small facilities. Many states will not approve use of this type of chlorinator.

2. Vacuum feed. This type of chlorinator has wide spread application and is much safer to use than direct feed.

Safety Equipment

The following safety equipment should be provided.

1. Exhaust fan located near floor level and switched to come on automatically when the chlorination room is entered. The fan should be of sufficient size to produce one air change per minute.

2. Emergency breathing apparatus located near the door or outside the chlorination room.

3. Emergency chlorine container repair kits.

4. Chlorine leak detection.

5. In most cases, alarms should be provided to alert the operator in the event of deficiencies or hazardous conditions.

Diffusers

Diffusers should be designed for a minimum velocity through diffuser holes of 10-12 feet per second and installed such that they can be easily removed for cleaning or replacement.

OPERATION AND MAINTENANCE

A properly designed chlorination facility will only perform as well as it is operated and maintained. Some important aspects of operation and maintenance are as follows:

1. Cleaning of contact chambers. Clean contact chambers reduce chlorine demand and thus operational costs significantly. Frequent cleaning is especially important in chlorination of pond effluents because solids can build up rapidly due to the settling of algae and other aquatic organisms.

2. Sampling and analysis. The amperometric method is probably the most accurate method of testing for chlorine residual. Frequent analysis should be made for chlorine residual and fecal coliform.

3. Chlorine dose. The chlorine dose required to produce the desired level of disinfection must be determined on a case-by-case basis. Since the amount of chlorine required varies drastically with pond effluents, studies should be conducted seasonally to identify changes in disinfection requirements. Dosage rates should be checked at least daily. The chlorine residual generally should be kept below 1 mg/l. This, however, must be determined on a case-by-case basis for each individual pond system.

4. Records. Periodic, preferably daily, records of chlorine dose, chlorine residual, coliform counts, pH, chlorine demand, etc., should be maintained.

5. Safety precautions. All operation and maintenance personnel should be trained in safety precautions involving chlorine.

6. Miscellaneous precautions. Operators must be alert to assure that there is continually an adequate supply of solution water, that chlorine injectors do not become plugged with algae, that strainers are frequently cleaned, and generally that all mechanical equipment is in good operation to assure the highest possible chlorination efficiency.

EXAMPLES OF STABILIZATION POND CHLORINATION

Variability in possible designs of chlorination facilities for stabilization ponds may best be pointed out by citing several examples.

Rigby, Idaho

The Rigby treatment facility basically consists of a four cell system. The first cell is aerated with two mechanical aerators. The ponds treat an average of about 0.25 mgd. Chlorination was added several years ago as required by the discharge permit. The chlorine contact chamber consists of a well baffled rectangular concrete basin as shown in Figure 8. The length/width ratio well exceeds the recommended design criteria of 40:1. A concrete scum baffle removes flotables prior to discharge. At peak flows, the detention time is 1 hour.

Chlorine dose is usually set at the amount of chlorine necessary to produce a residual of 0.2 mg/l at the end of the contact period. Under these conditions, chlorine dosage necessary to achieve the 0.2 mg/l residual is around 5 mg/l. Although a relatively low chlorine demand and excellent disinfection are achieved by this facility, there are several items that could be improved upon. For one, channels are too narrow for easy access for solids removal and cleaning of the contact basin. Fortunately, very little solids are accumulated, even in late summer when Daphnia blooms are a problem.

Another difficulty lies with the use of a direct feed chlorinator. On several occasions, chlorine gas has been detected being discharged directly to the atmosphere, creating a potential health hazard. Although a direct feed chlorinator is simple to operate, it does offer the serious disadvantage of potentially creating health hazards, particularly at facilities which are infrequently inspected. Also, the city has had some difficulty getting replacement parts and has found it advantageous to keep a good supply of spare parts in stock.

Roosevelt, Utah

The Roosevelt facility is a new pond system designed to treat 1.5 mgd. The facility basically consists of a four cell system. The last cell is a large winter storage reservoir which also serves as a final finishing pond. Effluent is stored throughout the winter and used to irrigate alfalfa in the summer with center pivot, sprinkler irrigation systems. Chlorination is required prior to land application.

The chlorine contact chamber consists of a 72" CMP. embedded in the embankment of the winter storage pond. This pressure conduit is designed for a contact time of 1 hour at peak pumping rates. Chlorination is accomplished by use of vacuum feed chlorinator fed by two 1-ton cylinders. Chlorine is fed from both cylinders simultaneously. Although this is not necessary, it does avoid having to use evaporators. The chlorinator feeds

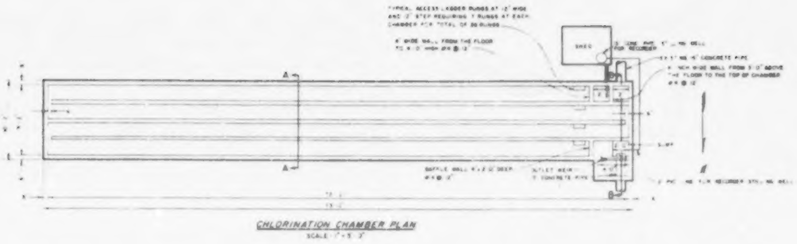


Figure 8. Rigby chlorine contact chamber.

at a rate sufficient for a dose of about 10 mg/l chlorine. Because of the newness of the facility, no major operational problems have been encountered to date.

Midvale, Utah

The Midvale facility is a 5 mgd aerated lagoon system. Aeration is accomplished by static tube diffused aerators. When chlorination became necessary, a nozzle type jet disinfection system manufactured by Penberthy was installed. In this type of system, chlorine is rapidly mixed with the effluent as it is forced through a jet type nozzle. The importance of mixing is emphasized in the system. The operator has found excellent disinfection with substantial savings of chlorine. He has estimated that the chlorine dose required is more than 50 percent less than it would have been with a conventional disinfection system. Currently, a chlorine dose of about 2 mg/l is being used to maintain a chlorine residual of 0.5-0.7 mg/l. This dosage is considerably less than the 6-7 mg/l being required at an adjacent trickling filter plant. The contact time necessary for adequate disinfection has also been reduced. At the Midvale plant, effluent is discharged into an outfall ditch following the jet disinfection and mixing system. The ditch provides 30 minutes of contact time for the effluent prior to discharging it into the receiving water. No major operational problems have been encountered to date and the chlorine demand has stabilized to a fairly constant level. Occasionally, chlorine demand will fluctuate in response to industrial wastes which are periodically discharged into the treatment facility. No increase in chlorine demand has been observed during times of high algae concentration.

SUMMARY

Chlorination of pond effluents is being required in many cases, to meet current discharge standards. With proper design, careful monitoring, and conscientious operation and maintenance, it is possible to efficiently achieve a desired level of disinfection while minimizing potential adverse effects. However, it must be cautioned that while many general design and operational criteria are applicable, pond effluent qualities are highly variable from pond-to-pond and actual operational control parameters must be established by comprehensive field study for each system.

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COST ESTIMATES FOR OXIDATION POND SYSTEMS

Michael F. Torpy*

I. INTRODUCTION

Cost data have been compiled from the literature, equipment manufacturers, and consultant engineers to approximate the component costs of small lagoon systems for various conditions. The conditions considered were usually evaluated for flowrates between 0.1 and 5.0 mgd and include changes in lagoon retention time, land costs, energy costs, and application rates for land application units. The total cost of each treatment unit of a lagoon system was usually segmented by defining capital, land, operation and maintenance (O&M), and energy costs. The cost data rely on some common assumptions reflected in the reported costs. The assumptions are:

1. All costs are based on 1978 prices.

2. Capital costs (which include installation) are annualized with a capital recovery factor based on an interest rate of 5-5/8 percent compounded for 20 years.

3. Capital equipment have no resale value.

4. Land resale value is identical to purchase price and is evaluated with a sinking fund factor at 5-5/8 percent interest for 20 years.

5. Labor costs are $9.00 per man-hour.

6. Crop revenues from land application systems are calculated as net profits at $40.00 per acre of land irrigated.

7. Pumping efficiencies vary from 61 percent in the lower flowrate ranges to 65 percent in flowrates greater than 1.4 mgd.

8. Peak flow is two times average flowrate.

II. COST OF LAGOON SYSTEM COMPONENTS

A. Pretreatment

Grit chamber and Bar Screens - The capital costs for preliminary treatment of small wastewater treatment plant influents are described in Figure 1. The operation and maintenance costs are estimated by:

1. Labor at 1 man-hour per day.
2. Energy at 0.25 KWH per 1000 gallons (1).

B. Oxidation Ponds

1. Natural Aeration - The costs for naturally aerated oxidation ponds have been evaluated for variations in 1) Storage season (or, conversely the application season), and 2) land value.

Table 1 indicates the area required for a range of average flowrates when the storage season varies from 3 to 7 months and assuming the maximum depth from the pond surface is 8 feet and the roads require an additional 15 percent of the pond design area. Buffer zones and pond lining were not considered. Tables 1 and 2 indicate the total land area and volume, respectively, required for naturally aerated lagoons with variations in storage season.

Figures 2-4 describe the approximate naturally aerated lagoon segment costs with variations in the application season. The application season refers to the length of time during a year when the lagoon effluents can be discharged and is equal (in months) to 12-storage season (in months).

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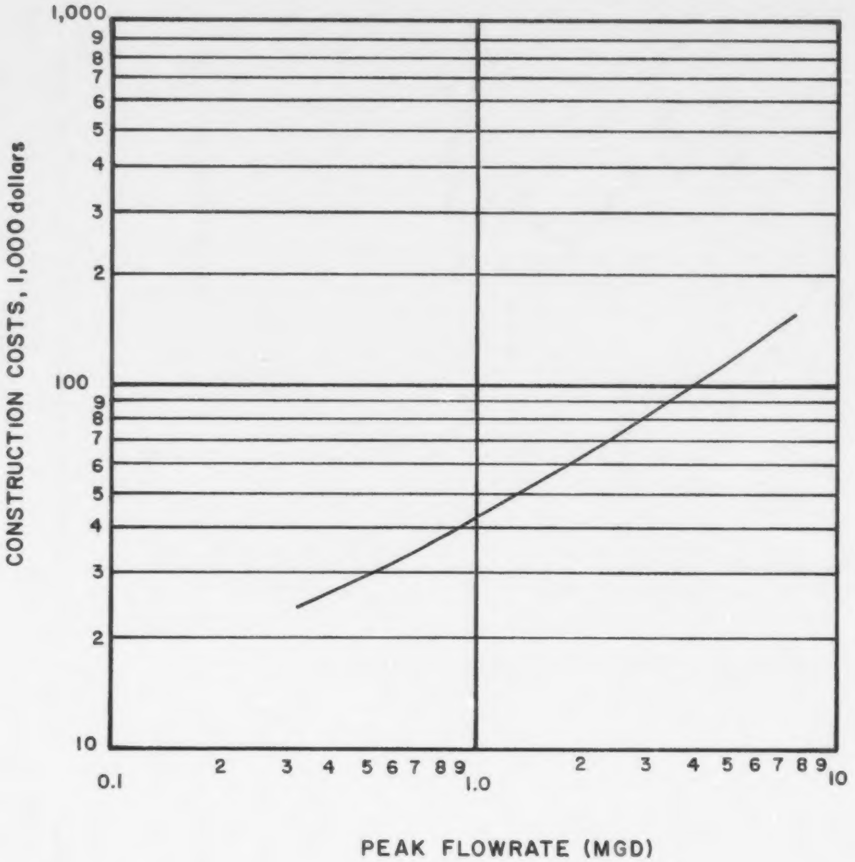

Figure 1. Capital costs for preliminary treatments (2).

Table 1. Naturally aerated lagoon area requirements.

Flow (mgd)	Storage Season (Months) 3	5	7
	Area (Acres)		
0.1	4	7	10
0.3	12	20	28
0.5	20	34	47
1.0	40	67	94
3.0	120	202	283
5.0	201	336	471

Table 2. Oxidation pond storage requirements.

Flow (mgd)	Storage Season (Months) 3	5	7
	Storage Volume (10^6 gal)		
0.1	9.1	15.2	21.3
0.3	27.4	45.6	63.8
0.5	45.6	76.0	106.4
1.0	91.2	152.1	212.8
3.0	273.8	456.3	638.4
5.0	456.3	760.4	1064.0

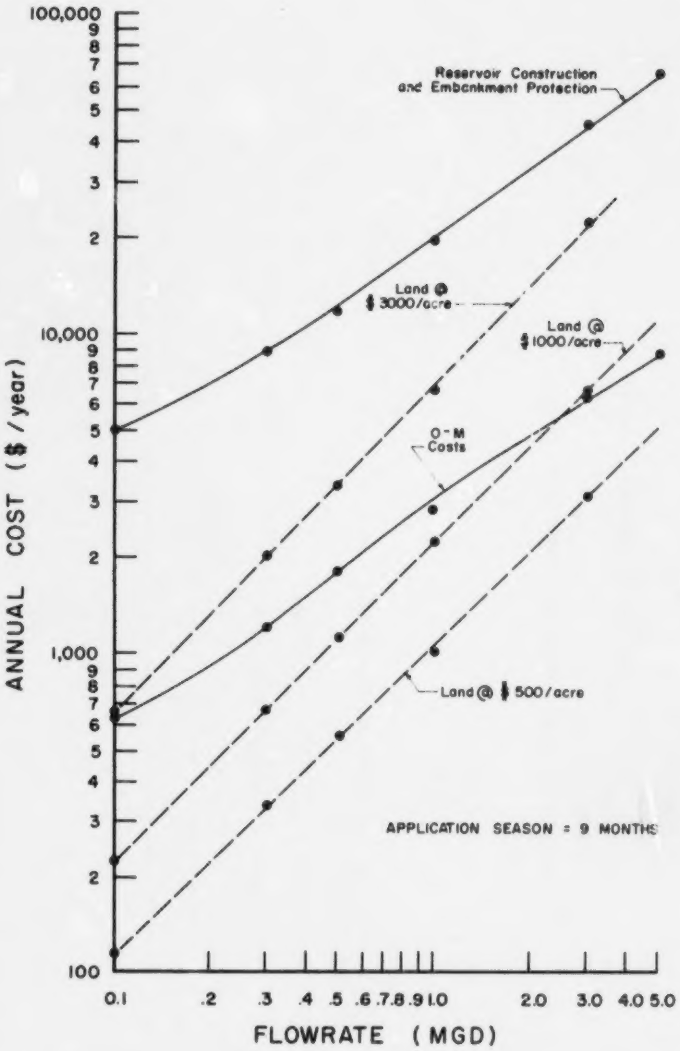

Figure 2. Flowrate versus annual costs for oxidation ponds.

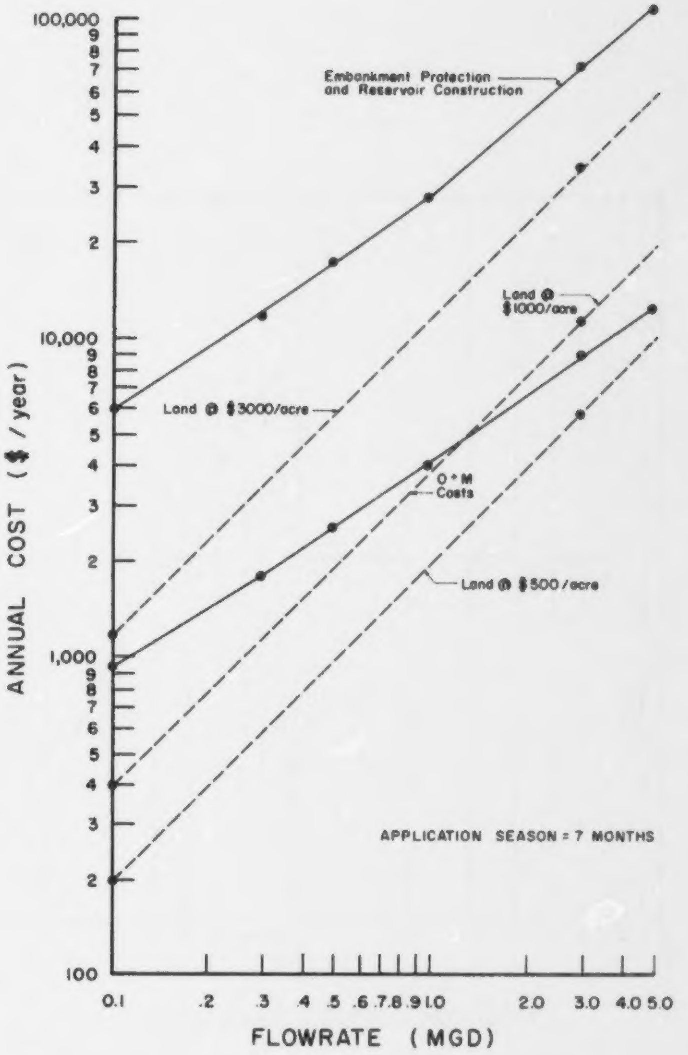

Figure 3. Flowrate versus annual cost for oxidation ponds.

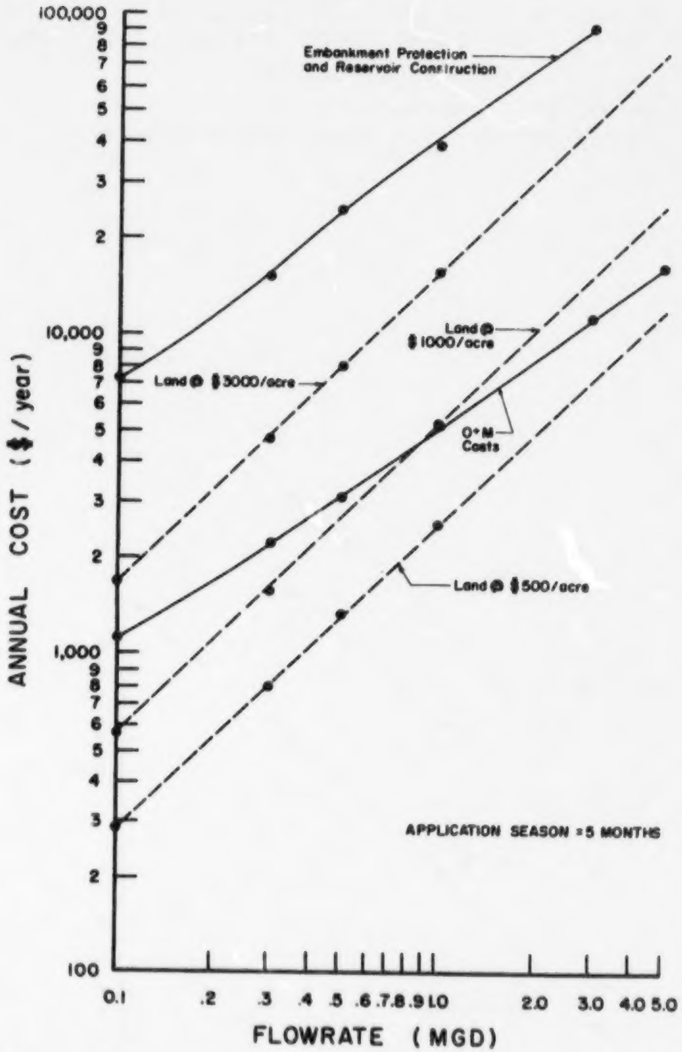

Figure 4. Flowrate versus annual cost for oxidation ponds.

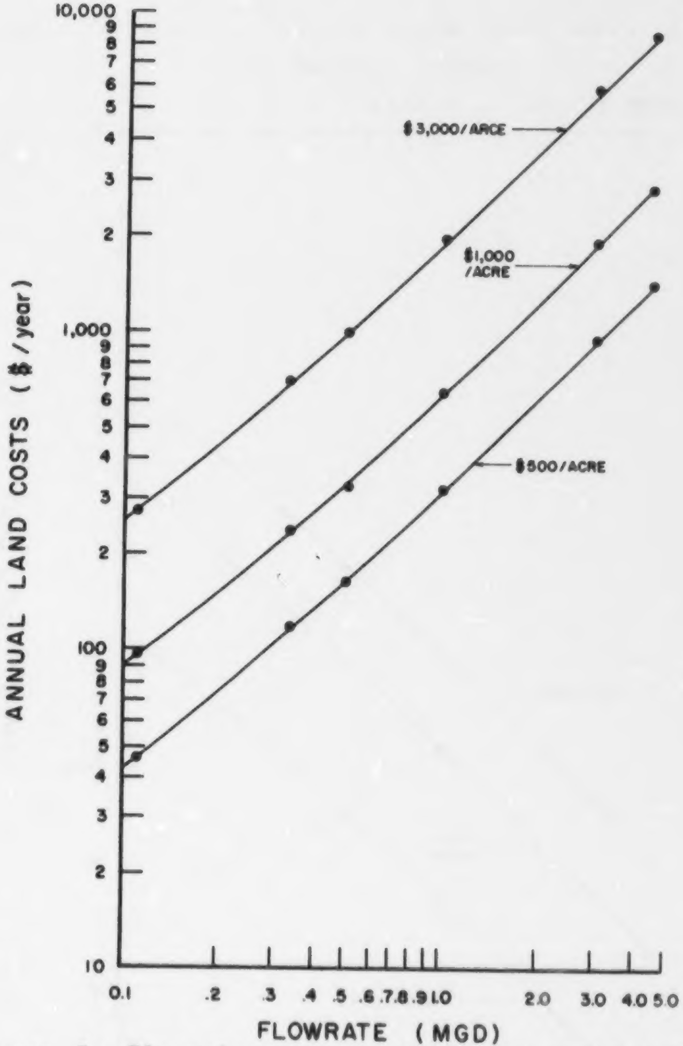

Figure 5. Flowrate versus annual land costs for submerged aeration ponds producing 85 percent BOD removal.

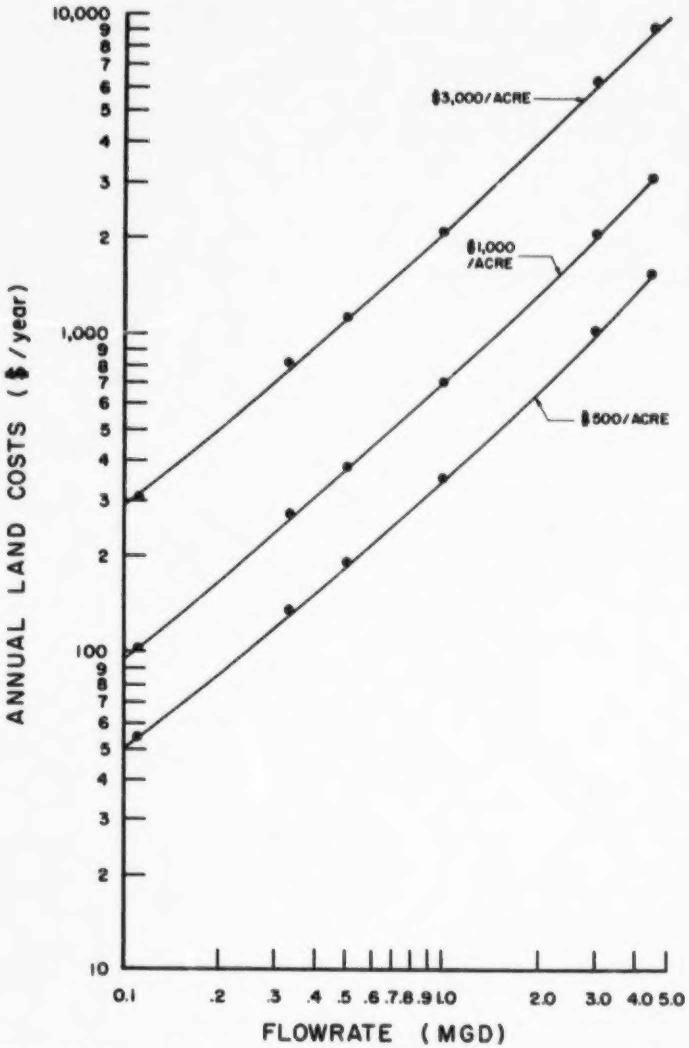

Figure 6. Flowrate versus annual land costs for submerged aeration ponds producing 90 percent BOD removal.

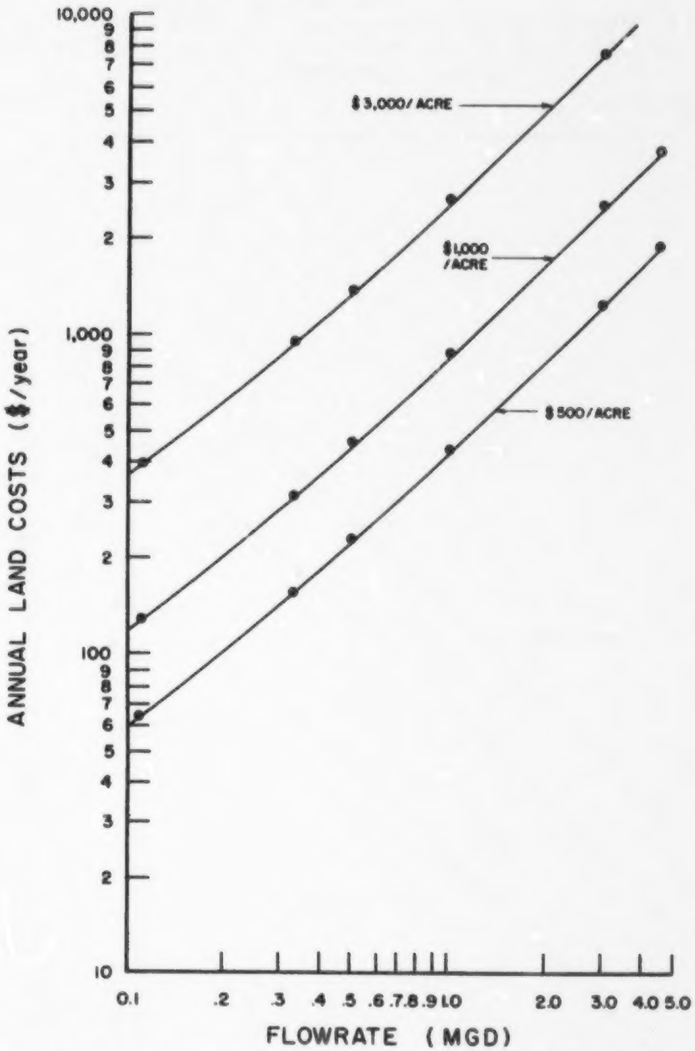

Figure 7. Flowrate versus annual land costs for submerged aeration ponds producing 95 percent BOD removal.

2. Submerged Aeration - The efficiency of BOD removal for submerged aeration in oxidation ponds varies with the detention time. The costs for submerged aeration were evaluated for 85, 90, and 95 percent BOD removal with the assumptions that one person contributes 250 lb of BOD per day and the pond depth is 10 feet. Figures 5, 6, and 7 indicate the land cost segment of the capital costs at 85, 90, and 95 percent BOD removal, respectively, when land costs vary at $500.00, $1,000.00, and $3,000.00 per acre. Figure 8 includes other segment costs for submerged aeration and apply for BOD removal at 85, 90, and 95 percent efficiency.

3. Surface Aeration - The costs of surface aeration ponds operating at an 85 percent BOD removal efficiency were calculated according to the design specifications listed in Table 3. The design and pump costs were provided by Aqua-Aerobic Systems, Inc., Rockford, IL (4). Other costs are updated costs from Pound et al. (3). A summary of the costs is described in Table 4.

C. Land Application Units

The use of land application units as an alternative to discharging lagoon effluents directly to a stream provides some definite advantages. Some considerations important to land application choices are described in Table 5 and the loading rates for alternative land application units are described in Figure 9 according to soil types.

The annual energy costs for land application units vary according to the pumping requirements for the unit. Figure 10 is provided to indicate the annual energy costs for different flowrate and total dynamic pumping head requirements assuming energy costs of $0.02/KWH and the application season is 9 months. To calculate costs for conditions that vary from a 9 month application season, the respective proportional values from Table 2 can be used.

Figure 11 indicates the land resource requirements for land application

Table 3. Surface aeration lagoon design for 85 percent BOD removal.

Flow (mgd)	1st Stage Lagoon				
	# of Lagoons	Depth (ft)	Surface Area (ft^2)	# of Aerators	Aerator Size (HP)
0.1	1	10	4,096	1	3.0
1.0	1	14	28,900	4	7.5
5.0	2	14	71,300/ea	16	10.0
	2nd Stage Lagoon				
	# of Lagoons	Depth (ft)	Surface[a] Area (ft^2)	# of Aerators	Aerator Size (HP)
	1	12	8,056	1	3.0
	1	14	70,300	4	7.5
	4	14	98,100/ea	24	7.5

[a] Includes 1 day retention time for final settling.

Table 4. Costs of surface aeration oxidation ponds for 85 percent BOD removal.

Flow (mgd)	Costs ($/year)					Population Equivalent Costs ($/person/year)
	Capital					
	Storage Ponds	Pumps	O&M	Energy	Total	
0.1	2,059	422	475	1,568	4,524	9.05
1.0	7,642	2,266	1,016	15,677	26,601	5.32
5.0	27,769	12,750	3,143	88,840	132,502	5.30

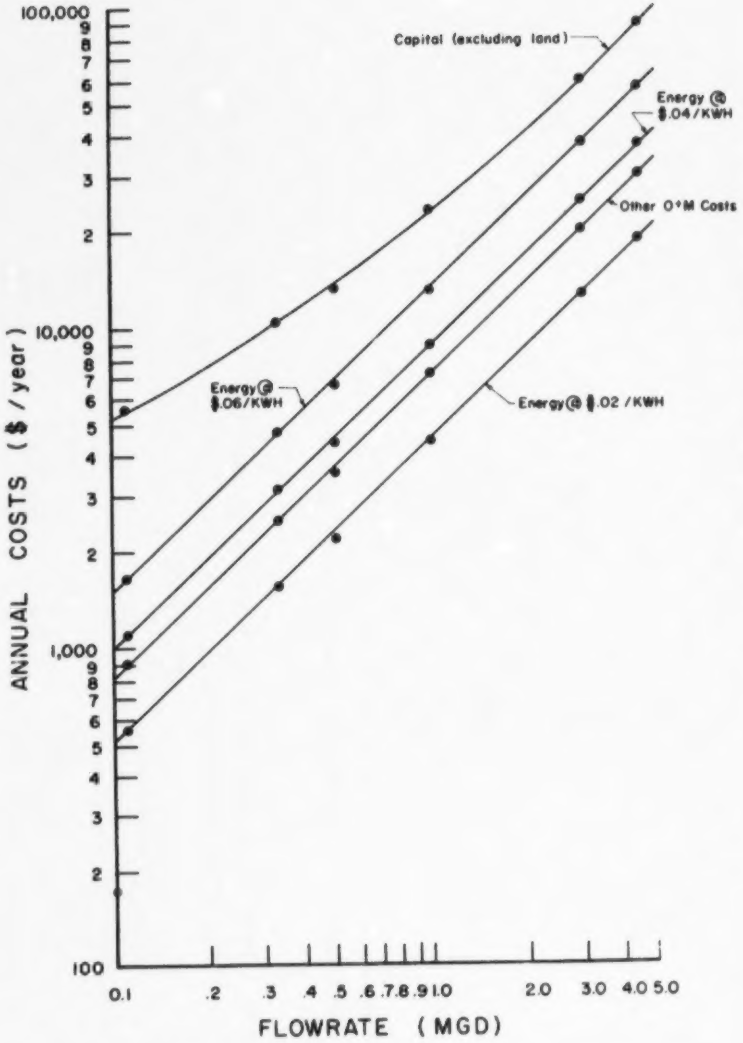

Figure 8. Flowrate versus annual costs for submerged aeration ponds.

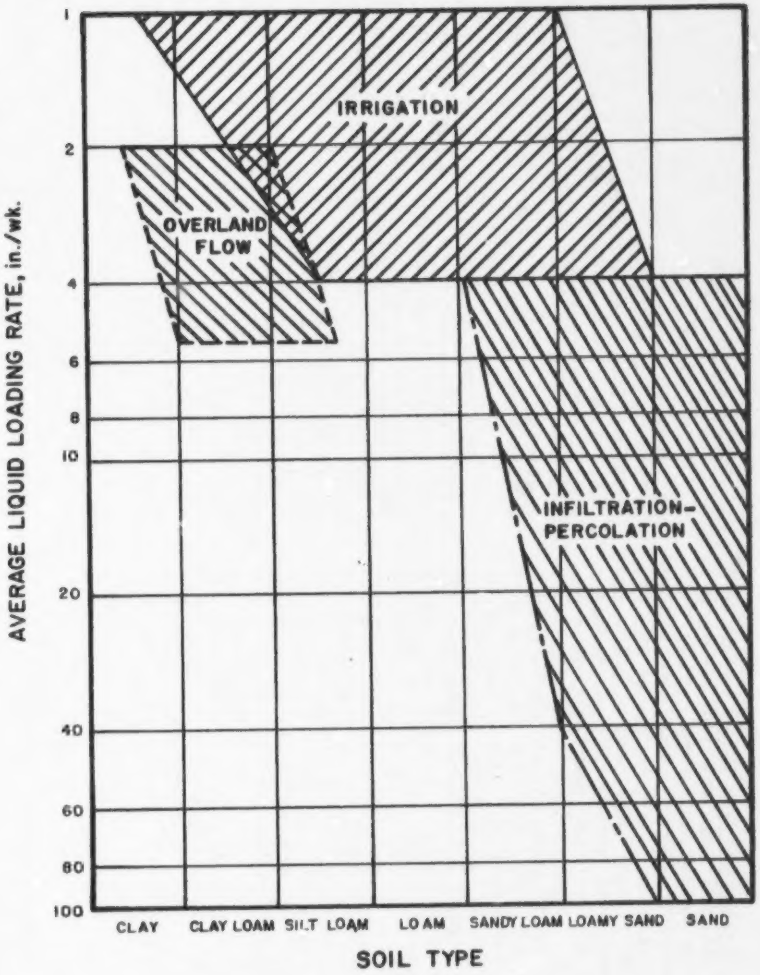

Figure 9. Soil type versus liquid loading rates for different land application approaches (5).

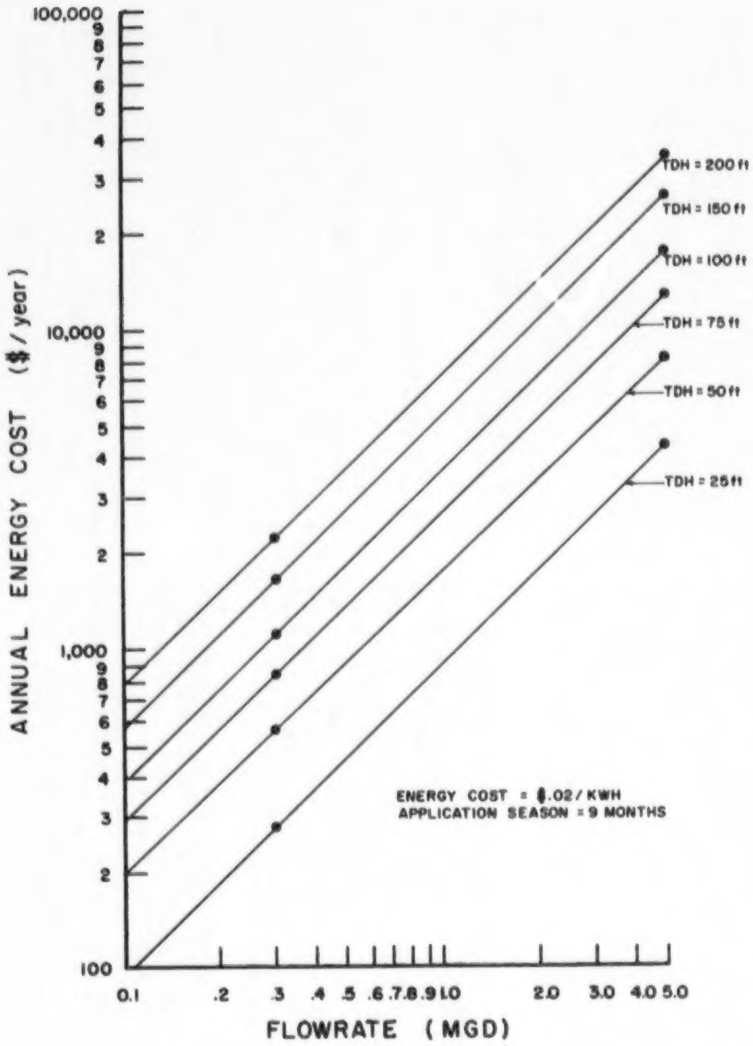

Figure 10. Flowrate and TDH versus annual energy cost for land application units.

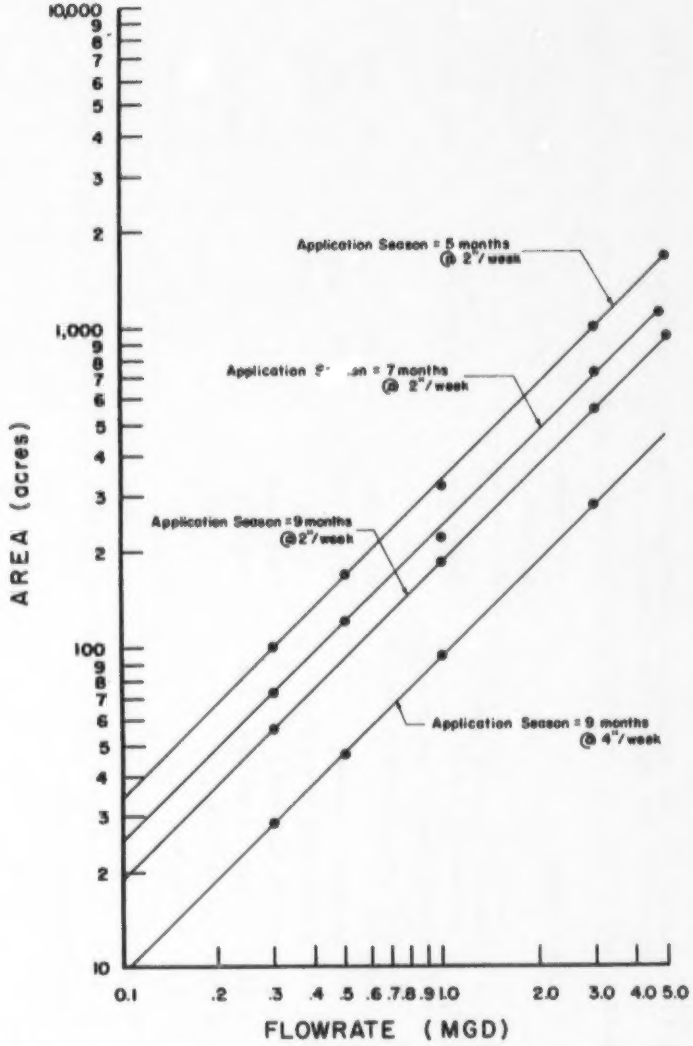

Figure 11. Flowrate and application rate versus land requirements for land application treatment units.

Table 5. Comparison of irrigation, overland flow, and infiltration-percolation for municipal wastewater (Pound et al., 1973) (9).

Objective	Type of Approach		
	Irrigation	Overland Flow	Infiltration-percolation
Use as a treatment process with a recovery of renovated water	Impractical	50 to 60% recovery	Up to 90% recovery
Use for treatment beyond secondary:			
1. For BOD and suspended solids removal	90 - 99%	90 - 99%	90 - 99%
2. For nitrogen removal	Up to 90%[a]	70 - 90%	0 - 80%
3. For phosphorus removal	80 - 99%	50 - 60%	70 - 95%
Use to grow crops for sale	Excellent	Fair	Poor
Use as direct recycle to the land	Complete	Partial	Complete
Use to recharge groundwater	0 - 30%	0 - 10%	Up to 90%
Use in cold climates	Fair[b]	---[c]	Excellent

[a] Dependent upon crop uptake.

[b] Conflicting data--woods irrigation acceptable, cropland irrigation marginal.

[c] Insufficient data.

units when the application season is varied from 5-9 months and the application rate is varied from 2 inches to 4 inches per week. The costs of four land application units; solid set sprinkler, center pivot, overland flow, and ridge and furrow, have been evaluated according to an application season of 9 months (storage season of 3 months), a land value of $500.00 per acre and energy costs of $0.02/KWH. Some of the segment costs are updated from Pound (3).

1. Solid Set Sprinkler - Figure 12 indicates the total annual costs for a solid set sprinkler unit for varying application seasons and rates with a total dynamic pumping head of 100 feet and energy cost of $0.02/KWH. Figures 13 and 14 are provided to describe the segment costs for capital, land, operation and maintenance, and energy can be used with Figure 12 to calculate total annual costs when conditions vary from those applied in Figure 12.

2. Center Pivot Spray Irrigation - Figure 15 indicates the total annual costs for center pivot spray irrigation assuming a total dynamic pumping head of 100 feet, land value of $500.00/acre and an energy cost of $0.02/KWH. Figures 16 and 17 are supplements for Figure 15 to provide a basis for calculating total annual costs under conditions that vary from those of Figure 15.

3. Overland Flow - Figure 18 is provided to estimate the total annual costs for overland flow with the assumptions that the total dynamic pumping head is 50 feet, land value is $500.00/acre and energy costs are $0.02/KWH. Figures 19 and 20 can be used to compliment Figure 18 under varying assumptions.

4. Ridge and Furrow - Figures 21, 22, and 23 are provided for estimating the total annual cost of ridge and furrow irrigation under various resource cost differences.

D. Polishing Units

1. Rock Media Filters - Cost estimates for rock media filters were provided after a unit with a liquid capacity of 0.6×10^6 gallons, designed at 3.81 gallons per day of inflow per ft^3 of rock media (6). The capital costs (including pumps and installation) are $66,000 and services a population equivalent of 0.6 mgd with a detention period of 24 hours and a large design safety factor. Operation and maintenance require 2 man hours per day with energy pumping requirements of about 11,000 KWH per year when operating continuously for 7 months. For BOD and suspended solids influents of 160-300 ppm, the removal efficiency is about 95 percent.

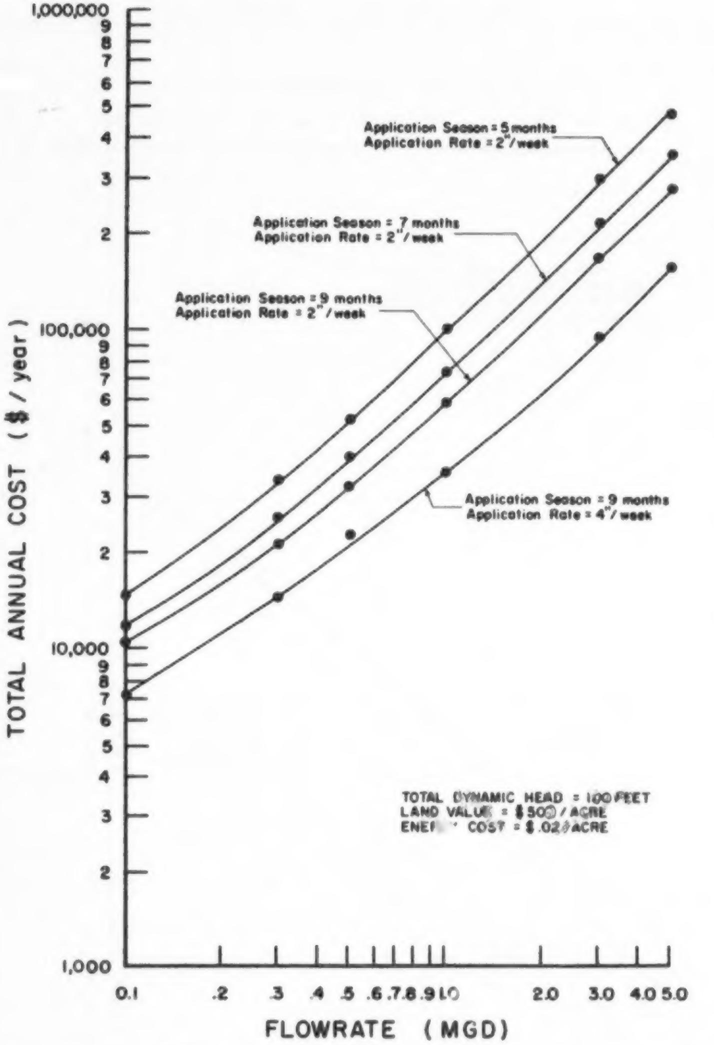

Figure 12. Flowrate, application season, and application rate versus total cost for a solid set sprinkler unit.

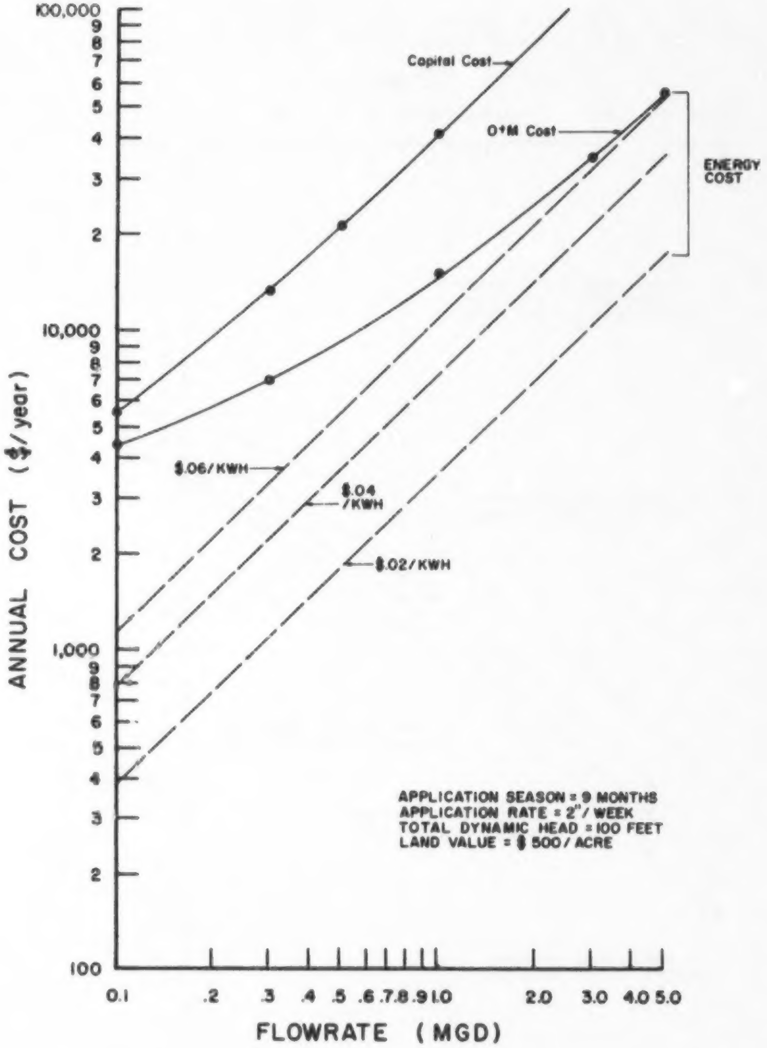

Figure 13. Flowrate versus cost for a solid set sprinkler unit.

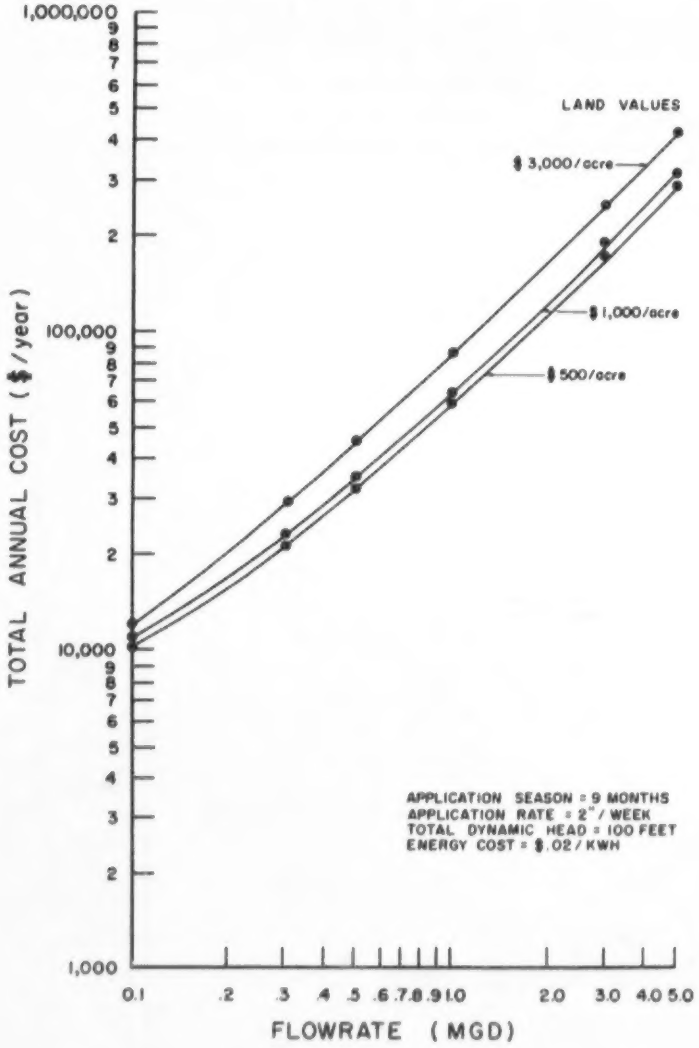

Figure 14. Flowrate and land value versus total cost for a solid set sprinkler unit.

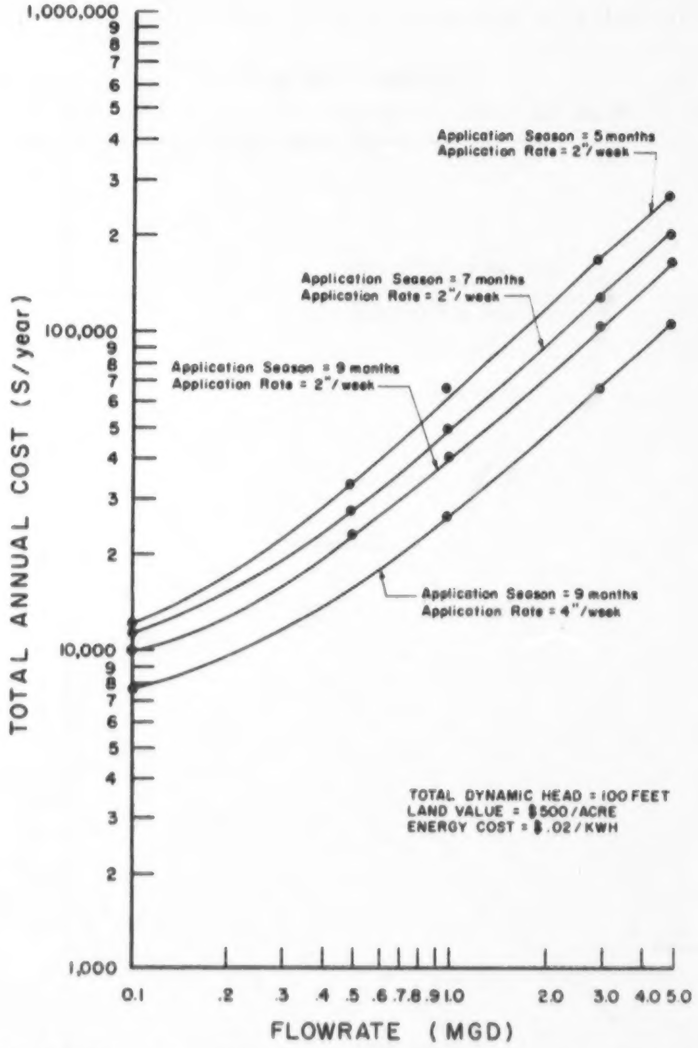

Figure 15. Flowrate, application season, and application rate versus total cost for a center pivot unit.

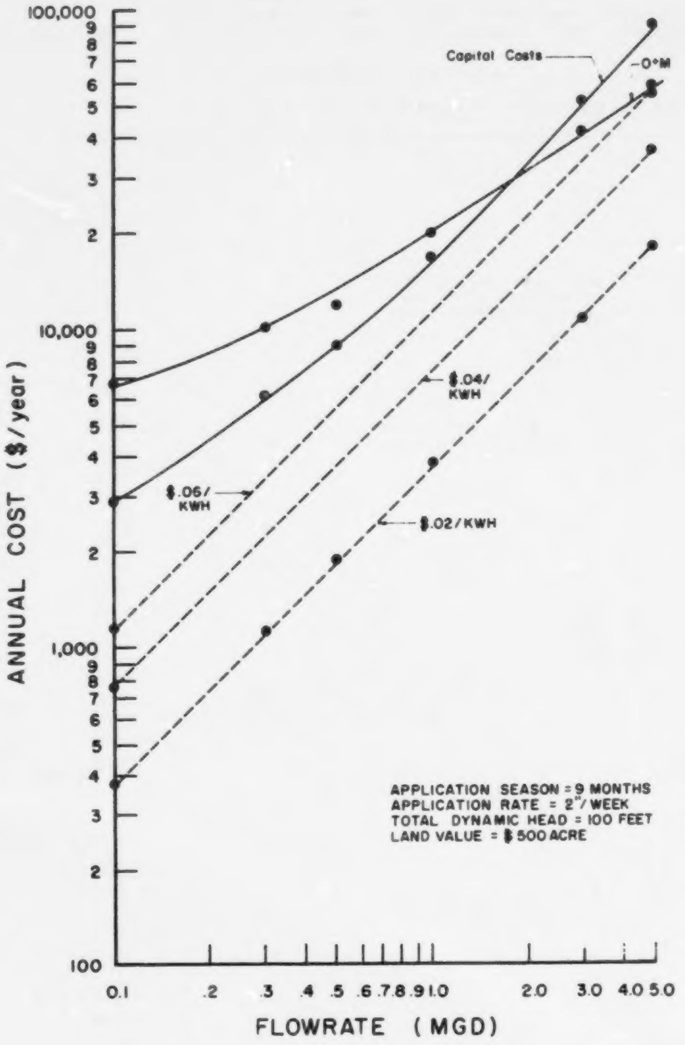

Figure 16. Flowrate versus costs for a center pivot unit.

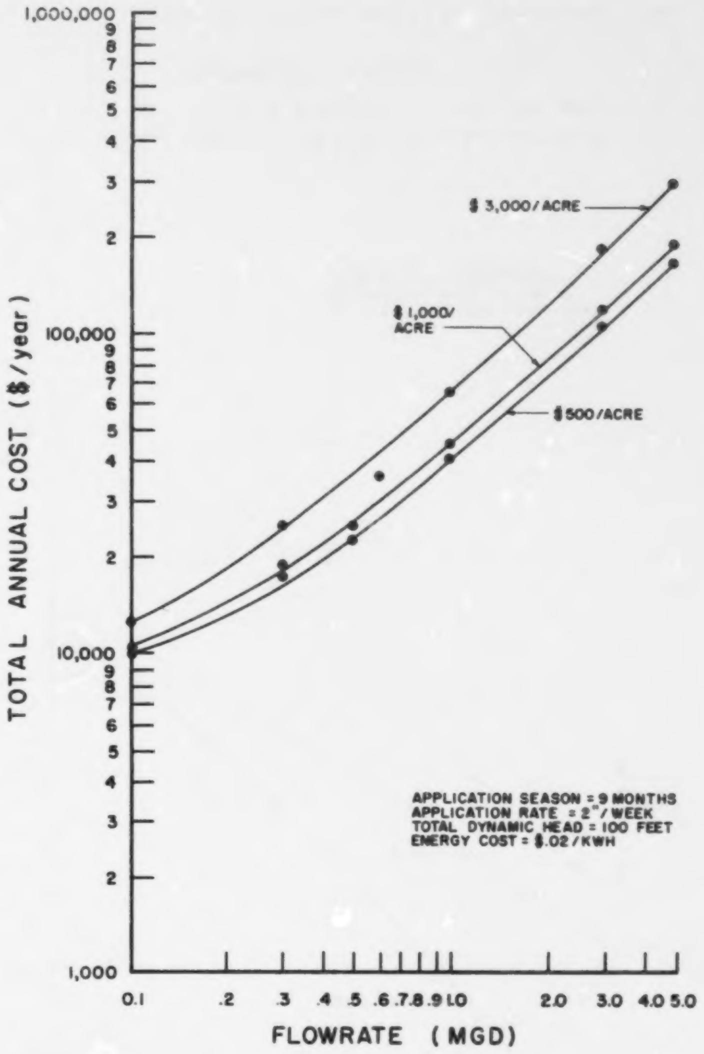

Figure 17. Flowrate and land value versus total cost for a center pivot unit.

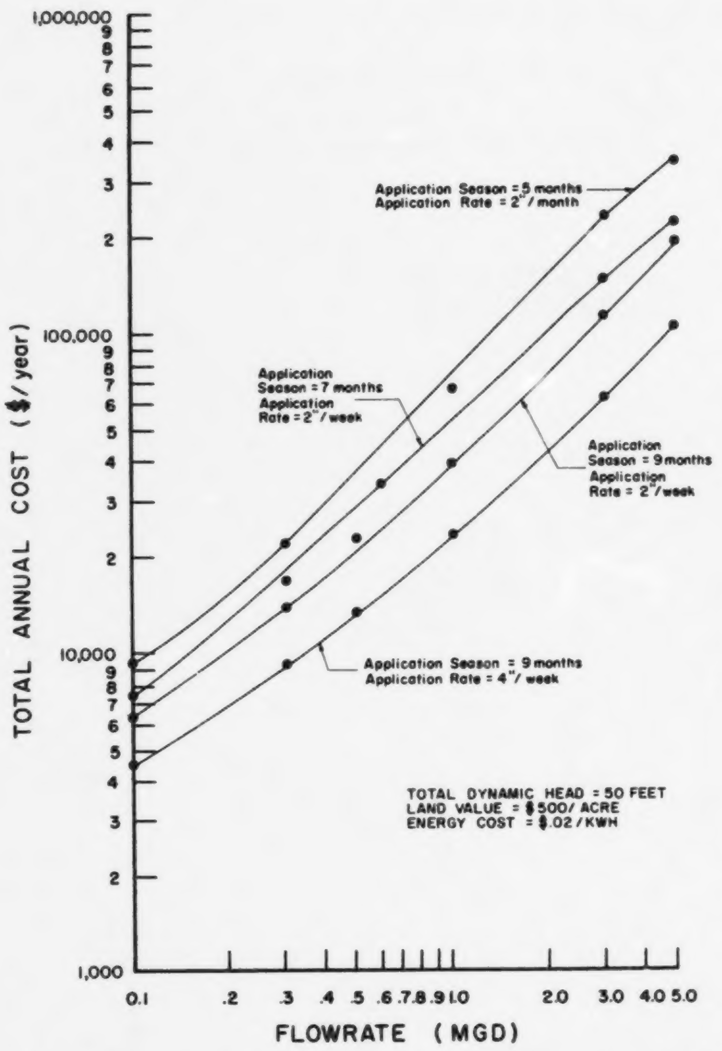

Figure 18. Flowrate, application season, and application rate versus total cost for overland flow units.

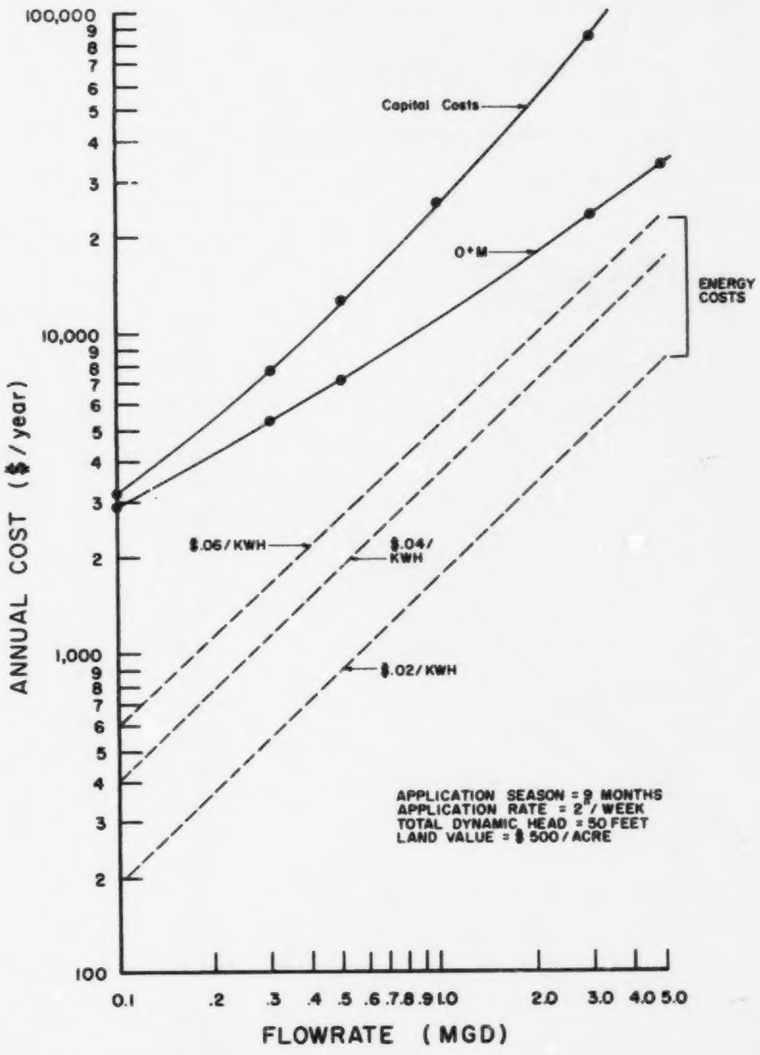

Figure 19. Flowrate versus cost for overland flow units.

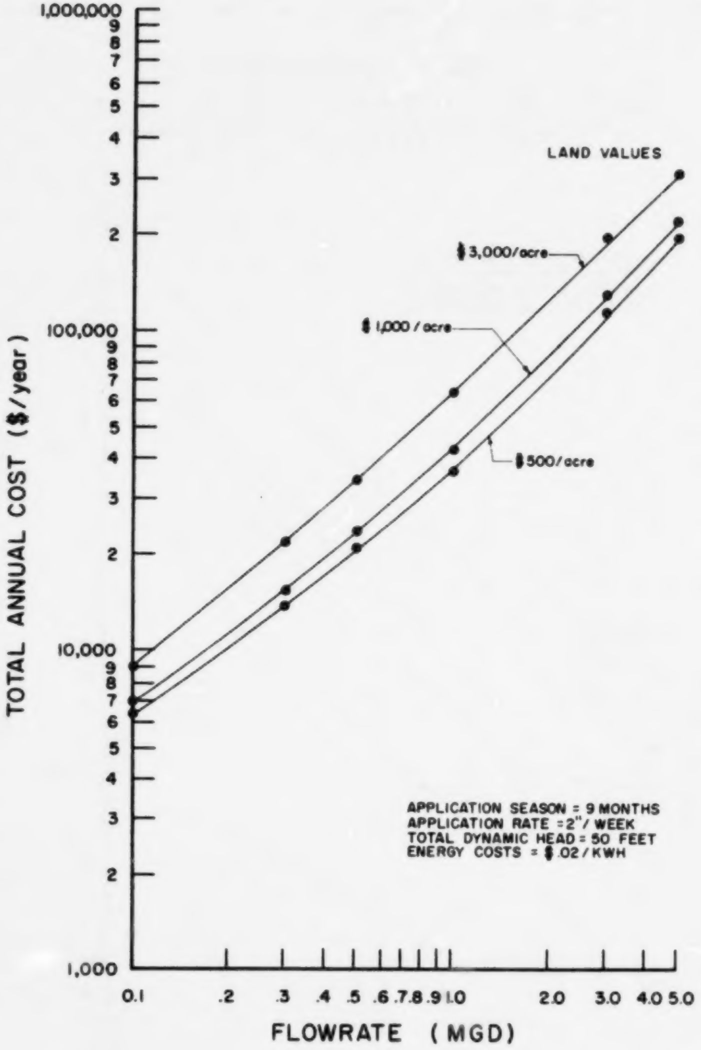

Figure 20. Flowrate and land value versus total cost for overland flow units.

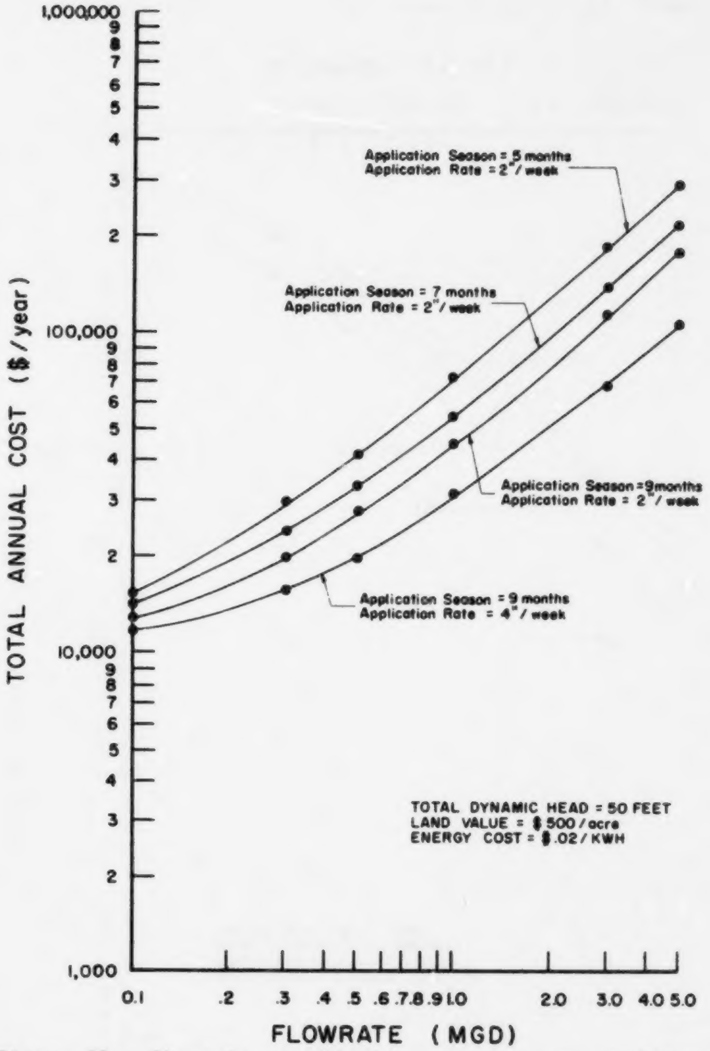

Figure 21. Flowrate, application season, and application rate versus total cost for ridge and furrow irrigation units.

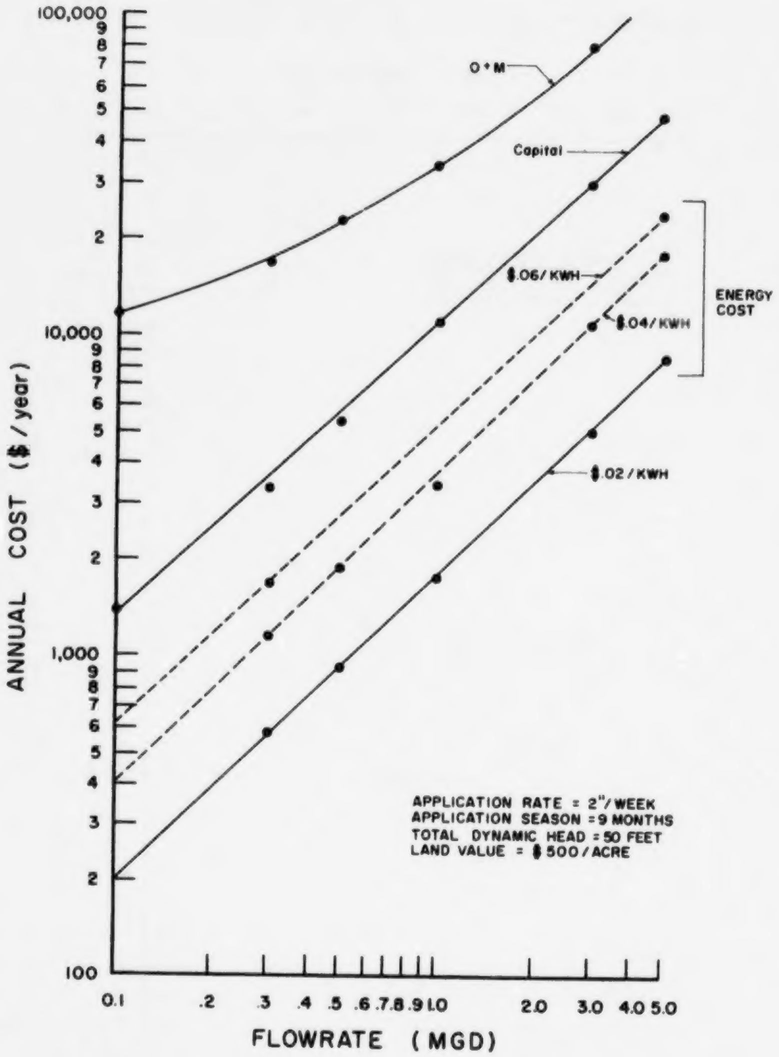

Figure 22. Flowrate versus cost for ridge and furrow irrigation units.

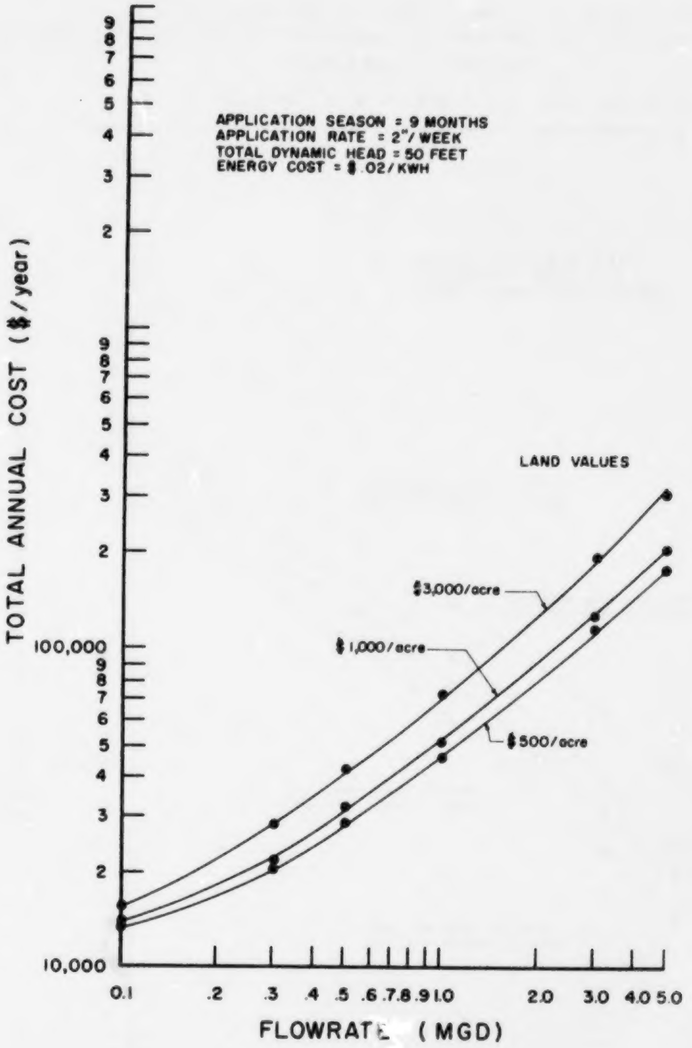

Figure 23. Flowrate and land value versus total cost for ridge and furrow irrigation units.

2. Microscreens - The annual costs for a 1.7 mgd microscreen unit (7) are listed below

	Annual Cost
Capital - \$595,000	\$50,307
O&M	19,000
Energy (196 x 10^3 KWH/yr) @ \$ .033/KWH	6,600
TOTAL	\$75,907

3. Dissolved Air Flotation - The annual costs for a dissolved air flotation unit with a 1.7 mgd capacity (7) are listed below

Capital - \$300,000	\$25,365
O&M - \$168,400	168,400
Energy (196 x 10^3 KWH/yr) @ \$ .033/KWH	6,600
TOTAL	\$200,365

4. Intermittent Sand Filters - The costs for intermittent sand filters are reported from the literature in 1973 dollars and listed below on the assumption that the filters are cleaned manually (8).

Capital - \$190,000	\$16,065
O&M (@ 2360 m-hour/yr)	18,880

E. Chlorination Unit

The chlorination costs for a range of flowrates are described in Figure 24, assuming a chlorine dose of 5 mg/l and a cost of \$0.10/lb for chlorine (3).

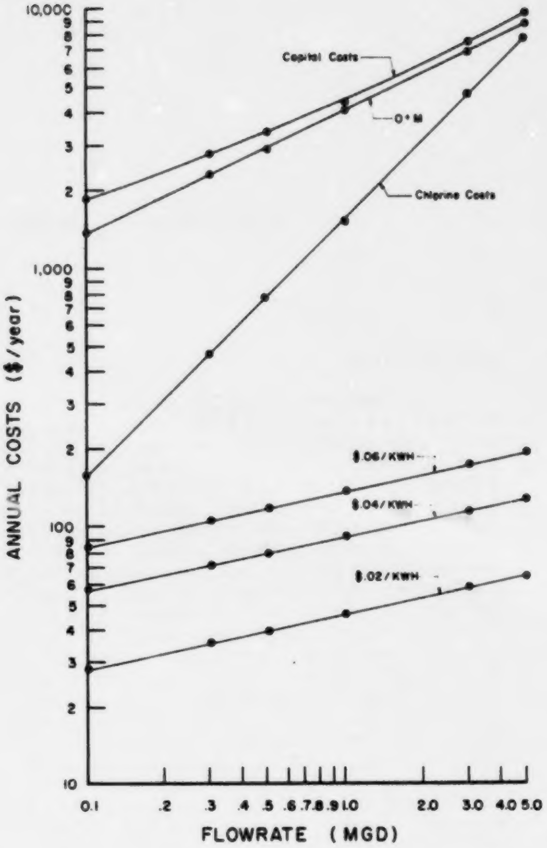

Figure 24. Flowrate versus annual costs for the chlorination of lagoon effluent.

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2. Benjes, H.H., Jr. 1978. Small Community Wastewater Treatment Facilities-Biological Treatment Systems. In Design Seminar Handout: Small Wastewater Treatment Facilities, EPA p. 98.

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5. Pound, C. E., and R. W. Crites. 1973. Wastewater Treatment and Reuse by Land Application - Vol. II. EPA (EPA-660/2-73-006b).

6. Walter, Ray V. 1968. Schaudt, Stem and Walter, Inc., Eugene, Oregon.

7. Kormanik, R. A. 1978. Envirex, Inc., Waukesha, Wisconsin.

8. Middlebrooks et al. 1978. Performance and Upgrading of Wastewater Stabilization Ponds. In Design Seminar Handout: Small Wastewater Treatment Facilities. USEPA.

PERFORMANCE OF AERATED WASTEWATER STABILIZATION PONDS

E. J. Middlebrooks
J. H. Reynolds
C. H. Middlebrooks*

INTRODUCTION

Wastewater stabilization ponds are effective in reducing BOD_5 and have only one basic disadvantage, i.e., high concentrations of solids in the effluents. These solids leave the lagoon along with the other constitutents and can create problems in receiving streams. Concentrations of suspended solids can exceed 100 mg/l, but, as shown in the detailed performance data presented herein, such high concentrations are usually limited to two to four months during the year. However, during some months of the year, the suspended solids concentration exceeds the standard specified by most regulatory agencies. With the new standards proposed in the September 2, 1976, issue of the Federal Register, small flow systems will be excluded from the suspended solids effluent requirements provided that these solids are in the form of algae. In areas where water quality limited streams occur, it is presumed that algae removal will be required. Interpreting what constitutes algae may create problems in many locations, and it may require extensive study to convince the regulatory agency that principally algae are being discharged.

The Environmental Protection Agency has produced excellent documents outlining the basic factors which need to be corrected in order to ensure proper design of wastewater stabilization ponds (EPA, 1973b; EPA, 1974). These documents may be obtained by writing to Technology Transfer, U.S. Environmental Protection Agency. Further discussion of design of wastewater stabilization ponds will be omitted from this paper because of the extensive amount of information available through Technology Transfer and elsewhere.

To satisfy the need for reliable lagoon performance data, in 1974 the U.S. Environmental Protection Agency sponsored four intensive facultative lagoon performance studies and five intensive aerated lagoon performance studies. The four facultative lagoon performance studies were described in the paper by Finney and Middlebrooks presented in these proceedings. The aerated lagoons were located at Bixby, Oklahoma (Reid, 1977), Pawnee, Illinois (Gurnham & Assoicates, Inc., 1976), GulfPort, Mississippi (Englande, 1975), Lake Koshkonong, Wisconsin (Polkowski, 1977), and Windber, Pennsylvania. Data collection encompassed 12 months with four separate 30 consecutive day sampling periods once each season.

Aerated waste stabilization ponds are medium depth, man-made basins designed for the biological treatment of wastewater. A mechanical aeration device is used to supply supplemental oxygen to the system. In general, an aerated lagoon is aerobic throughout its entire depth. The mechanical aeration device may cause turbulent mixing (i.e., surface aerator) or may produce laminar flow conditions (diffused air systems).

Although the development, history, and design of aerated wastewater stabilization ponds have been reported by several investigators (Boulier and Atchinson, 1974; Bartsch and Randall, 1971; McKinney, 1970) very little reliable year-round performance data was available until the U.S. Environmental Protection Agency funded the evaluation of five aerated lagoon systems (Lewis, 1977). A portion of the data from these studies will be presented in the following paragraphs.

SITE DESCRIPTION

Bixby, Oklahoma

A diagram of the Bixby, Oklahoma, aerated lagoon system is shown in Figure 1 (Reid, 1977). The system consists of

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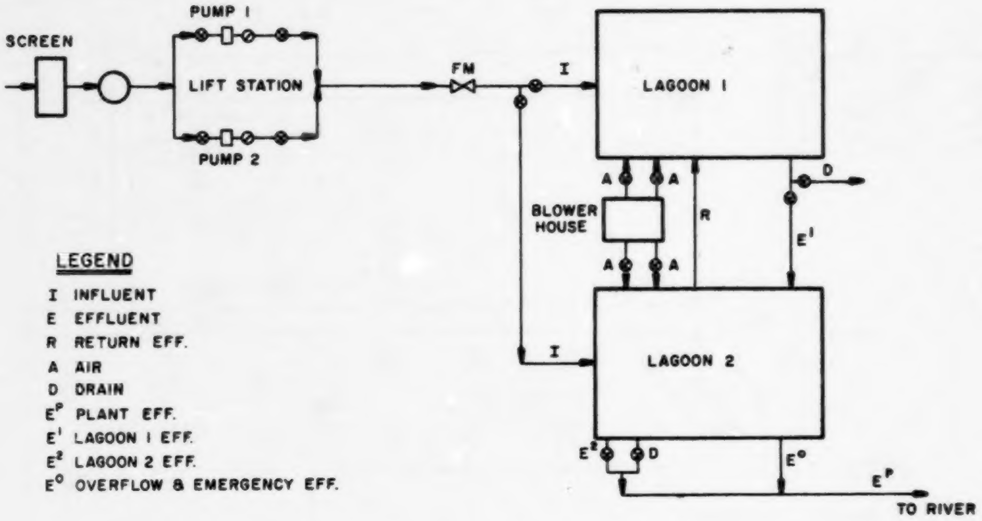
SCREEN
PUMP 1
LIFT STATION
PUMP 2
FM
I
LAGOON 1
A
BLOWER HOUSE
R
D
E'
I
LAGOON 2
E²
D
E°
E^P
TO RIVER
LEGEND
I INFLUENT
E EFFLUENT
R RETURN EFF.
A AIR
D DRAIN
E^P PLANT EFF.
E' LAGOON 1 EFF.
E² LAGOON 2 EFF.
E° OVERFLOW & EMERGENCY EFF.

Figure 1. Aerated lagoon system at Bixby, Oklahoma (Reid, 1977).

two aerated cells with a total surface area of 2.3 ha (5.8 acres). It was designed to treat 336 kg BOD_5/day (740 lbs BOD_5/day) with a hydraulic loading rate of 1551 m^3/day (0.4 mgd). There is no chlorination facility at the site. The hydraulic retention time is 67.5 days.

Pawnee, Illinois

A diagram of the Pawnee, Illinois, aerated lagoon system is shown in Figure 2 (Gurnham and Associates, Inc., 1976). The system consists of three aerated cells in series with a total surface area of 4.45 ha (11.0 acres). The design flowrate was 1893 m^3/day (0.5 mgd) with an organic load of 386 kg BOD_5/day (850 lbs BOD_5/day) and a theoretical hydraulic retention time of 60.1 days. The facility is equipped with chlorination disinfection and a slow sand filter for polishing the effluent. Data reported below were collected prior to the filters and represent only lagoon performance.

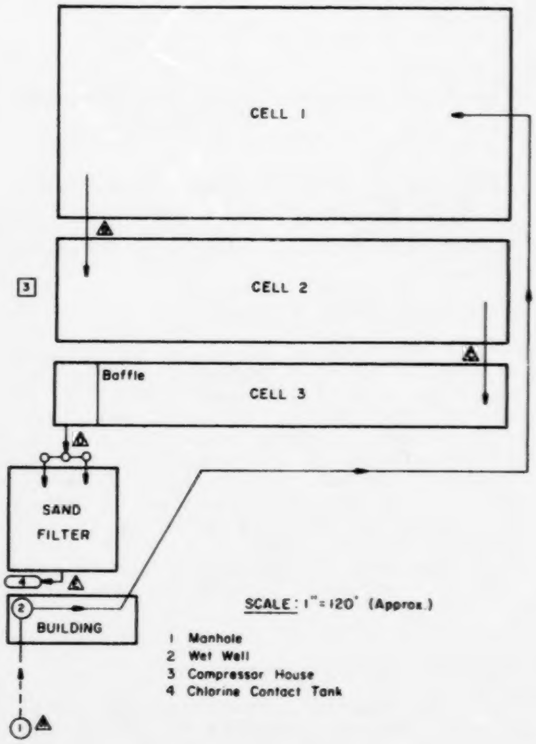
CELL 1
3
CELL 2
Baffle
CELL 3
SAND FILTER
4
2
BUILDING
1
SCALE: 1"=120' (Approx.)
1 Manhole
2 Wet Well
3 Compressor House
4 Chlorine Contact Tank

Figure 2. Aerated lagoon system at Pawnee, Illinois (Gurnham and Associates, Inc., 1976).

Gulfport, Mississippi

A diagram of the Gulfport, Mississippi, aerated lagoon system is shown in Figure 3 (Englande, 1975). The system consists of two aerated lagoons in series with a total surface area of 2.5 ha (6.3 acres). The system was designed to treat 1893 m^3/day (0.5 mgd) with a total theoretical hydraulic retention time of 26.2 days. The organic load on the first cell in the series was 374 kg BOD_5/

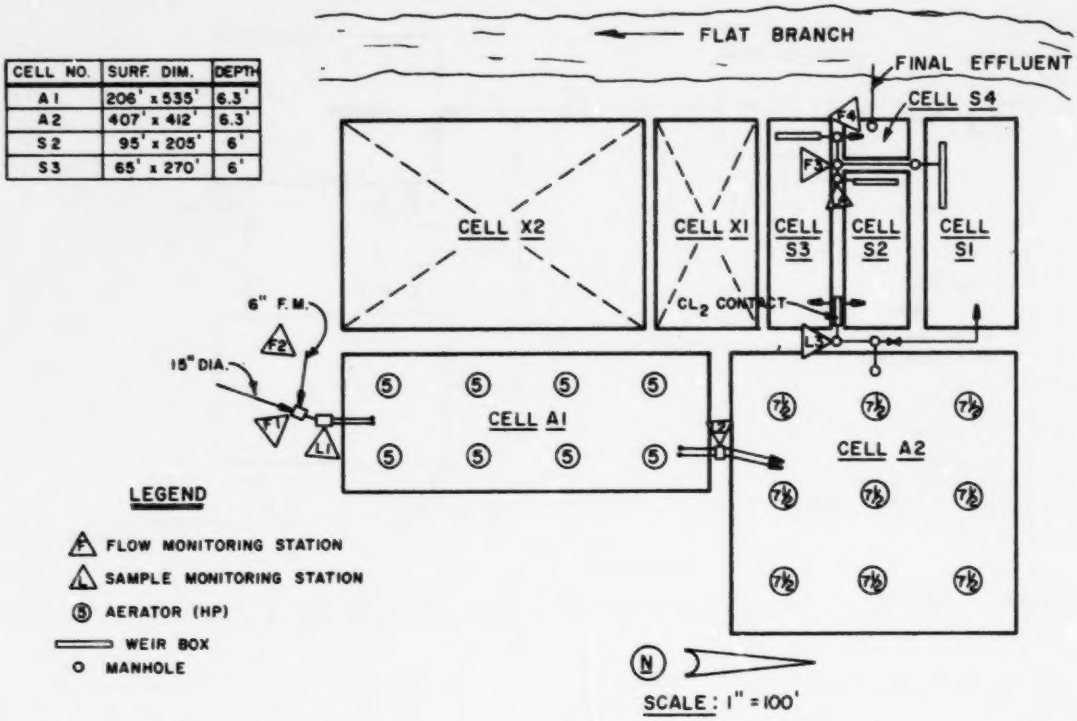
CELL NO.
SURF. DIM.
DEPTH
A1 206' x 535' 6.3'
A2 407' x 412' 6.3'
S2 95' x 205' 6'
S3 65' x 270' 6'
FLAT BRANCH
FINAL EFFLUENT
CELL S4
CELL X2
CELL X1
CELL S3
CELL S2
CELL S1
F4
F3
L3
CL2 CONTACT
6" F.M.
F2
15" DIA.
F1
L1
CELL A1
L2
CELL A2
5
7½
LEGEND
F FLOW MONITORING STATION
L SAMPLE MONITORING STATION
5 AERATOR (HP)
WEIR BOX
O MANHOLE
N
SCALE: 1" = 100'

Figure 3. Aerated lagoon system at Gulfport, Mississippi (Englande, 1975).

day/ha (334 lbs BOD_5/day/acre) and 86 kg BOD_5/day/ha (77 lbs BOD_5/day/acre) on the second cell in the series. The system is equipped with a chlorination facility.

Lake Koshkonong, Wisconsin

A diagram of the Lake Koshkonong, Wisconsin, aerated lagoon system is shown in Figure 4 (Polkowski, 1977). The system consists of three aerated cells with a total surface area of 2.8 ha (6.9 acres) followed by chlorination. The design flow was 2271 m^3/day (0.6 mgd) with a design organic load of 467 kg BOD_5/day (1,028 lbs BOD_5/day). The current organic loading rate is 248 kg BOD_5/day (545 lbs BOD_5/day) with a theoretical hydraulic retention time of 57 days.

Windber, Pennsylvania

The Windber, Pennsylvania, aerated lagoon system consists of three cells with a total surface area of 8.4 ha (20.7 acres) followed by chlorination. The design flow rate was 7,576 m^3/day (2.0 mgd) with a design organic loading rate

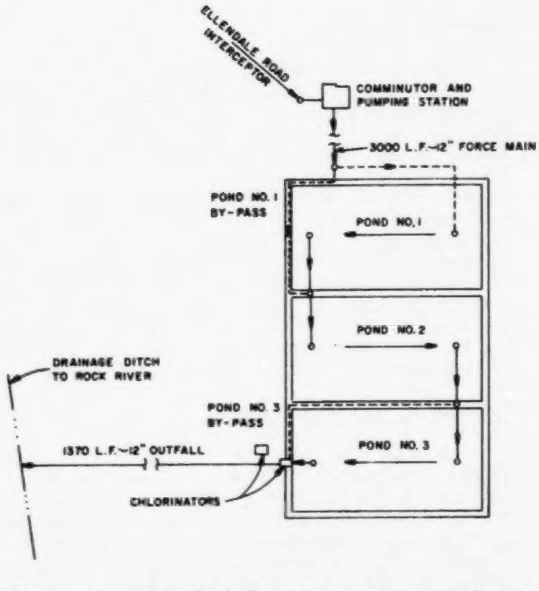
ELLENDALE ROAD INTERCEPTOR
COMMINUTOR AND PUMPING STATION
3000 L.F.-12" FORCE MAIN
POND NO. 1 BY-PASS
POND NO. 1
POND NO. 2
DRAINAGE DITCH TO ROCK RIVER
POND NO. 3 BY-PASS
1370 L.F.-12" OUTFALL
POND NO. 3
CHLORINATORS

Figure 4. Aerated lagoon system at Lake Koshkonong, Wisconsin (Polkowski, 1977).

of approximately 1,369 kg BOD_5/day (3,000 lbs BOD_5/day). The design mean hydraulic residence time was 30 days for the 3 cells operating in series. Actual influent flow rates varied from 3,000 to 5,300 m^3/day (0.8 to 1.4 mgd), the actual organic loading rate was approximately 924 kg BOD_5/day (2,030 lbs BOD_5/day), and the theoretical mean hydraulic residence time was approximately 55 days.

PERFORMANCE

Biochemical Oxygen Demand (BOD_5)

The monthly average effluent biochemical oxygen demand (BOD_5) removal for the five previously described aerated lagoon systems is reported in Table 1. The monthly average effluent BOD_5 concentrations are compared with the

Table 1. Performance summary of aerated lagoons.

Date (No. Sampling Days)	Description	Biochemical Oxygen Demand (mg/l)		Suspended Solids (mg/l)		Geometric Mean Fecal Coliform (No/100 ml)	
		Influent	Effluent	Influent	Effluent	Influent	Effluent
PAWNEE, ILLINOIS							
March, 1976 (7)	Q = 1893 m^3/day	233	3	178	22	-	-
April (30)	Surface area	277	3	236	10	-	8
May (7)	= 4.45 ha	470	3	544	23	-	50
June (7)	3 cell system	470	6	370	52	-	70
July (31)	Chlorination	452	3	768	7	-	235
August (7)	provided	602	2	758	8	-	210
September (7)	Slow Sand Filter	578	2	529	21	-	3
October (31)	provided.	799	2	678	25	-	44
November (7)	Performance	548	2	560	14	-	5
December (7)	data represent	554	3	543	24	-	6
January (0)	only lagoon	-	-	-	-	-	-
February (7)	performance.	395	4	387	19	-	2
March (30)		296	10	417	29		<1
Average		473	4	497	21		15
GULFPORT, MISSISSIPPI							
December, 1975 (7)	Q = 1893 m^3/day	178	24	223	42	-	509
January, 1976 (30)	Surface area	214	26	291	32	-	169
February (7)	= 2.5 ha	199	20	194	47	-	124
March (7)	2 cell system	192	25	195	43	-	1,222
April (30)	Chlorination	178	26	172	34	-	2,708
May (7)	provided	175	21	145	38	-	2,456
June (7)		171	23	272	38	-	2,687
July (30)		134	20	191	33	-	1,193
August (7)		151	25	122	30	-	10,782
September (30)		170	30	146	19		16,259
October (7)		171	34	117	16		21,638
November (7)		186	27	172	36		58,745
Average		177	25	187	34		4,802
WINDBER, PENNSYLVANIA							
November, 1975 (7)	Q = 3407 m^3/day	177	1	186	2	-	2
December (7)	3 cell system	203	5	178	4	-	2
January (7)		152	9	121	13	-	3
February (29)		220	10	107	10	-	2
March (8)		186	9	124	15	-	6
April (7)		155	5	128	6	-	2
May (29)		201	10	167	13	-	2
June (7)		182	17	158	16	-	2
July (27)		145	26	149	10	-	2
August (11)		162	16	182	10	-	2
September (7)		173	4	177	7	-	3
October (30)		106	17	151	4	-	3
Average		172	11	152	9	-	3
BIXBY, OKLAHOMA							
January, 1976 (24)	Q = 1552 m^3/day	368	48	323	51	-	-
February (7)	Surface area	504	41	199	46	-	-
March (7)	= 2.3 ha	460	53	236	85	11,042	4,669
April (25)	2 cell system	402	36	314	63	-	-

Table 1. Continued.

Date (No. Sampling Days)	Description	Biochemical Oxygen Demand (mg/1)		Suspended Solids (mg/1)		Geometric Mean Fecal Coliform (No/100 ml)	
		Influent	Effluent	Influent	Effluent	Influent	Effluent
May (9)	No chlorination	448	36	250	57	-	-
June (7)	provided	-	-	230	96	-	-
July (21)		355	20	254	71	-	-
August (9)		212	14	282	38	-	-
September (3)		330	40	213	28	-	-
October (6)		388	19	230	51	-	-
November (19)		383	24	289	36	-	-
Average		388	32	256	56	-	-
LAKE KOSHKONONG, WISCONSIN							
December, 1975 (7)	$Q = 2271\ m^3/day$	89	8	184	4	-	-
January, 1976 (30)	Surface area	96	10	115	3	-	-
February (7)	= 2.3 ha	73	5	123	2	-	-
March (7)	3 cell lagoon	37	11	48	26	-	-
April (30)	system	55	10	64	22	-	-
May (7)	Chlorination	71	13	64	55	-	-
June (7)	provided	86	11	100	19	-	-
July (30)		93	20	133	23	-	-
August (7)		118	19	183	16	-	-
September (7)		113	17	126	21	-	-
October (30)		102	11	114	8	-	-
November (7)		87	4	116	5	-	-
Average		85	12	110	17	-	-

Federal Secondary Treatment Standard of 30 mg/l in Figure 5.

In general, all of the systems studies, except the Bixby, Oklahoma, system, were capable of producing a final effluent BOD_5 concentration of 30 mg/l. Average monthly effluent BOD_5 concentrations appear to be independent of influent BOD_5 concentra-

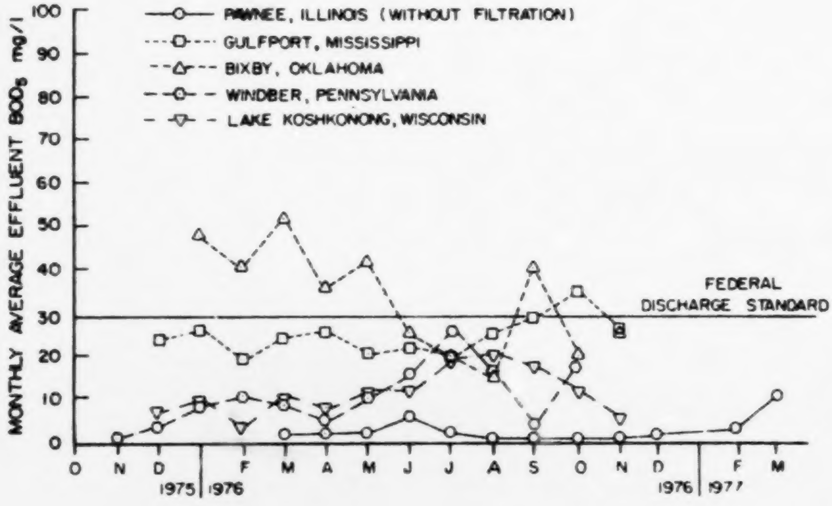

Figure 5. Aerated lagoon Biochemical Oxygen Demand (BOD_5) removal performance.

tion fluctuations and are also not significantly affected by seasonal variations in temperature.

Average monthly influent BOD_5 concentrations at Bixby, Oklahoma, ranged from 212 mg/l to 504 mg/l with an average of 388 mg/l during the study period reported. The design influent BOD_5 concentration was 240 mg/l, or only 62 percent of the actual influent concentration. The mean flow rate during the period of study was 523 m^3 day (0.123 mgd) which is less than one third of the design flow rate. The Bixby system was designed to treat 336 kg BOD_5/day (740 lbs BOD_5/day), and apparently a load of only 203 kg BOD_5/day (446 lbs BOD_5/day) was entering the lagoon. The only major difference between the Bixby and other aerated lagoons is the number of cells. Bixby has only 2 cells in series. Based upon the results of studies with facultative lagoons which show improved performance with an increase in cell number, this difference in configuration could account for the relatively poor performance by the Bixby system. However, there are many other possible explanations, i.e., operating procedures, inadequate air supply, short circuiting (related to number of cells in series), etc.

The results of these studies indicate that aerated lagoons which are properly designed, operated, and maintained can consistently produce an effluent BOD_5 concentration of less than 30 mg/l. In addition, effluent quality is not seriously affected by seasonal climate variations.

Suspended Solids Removal Performance

The monthly average suspended solids removal performance of the five previously described aerated lagoon systems is reported in Table 1. The monthly average effluent suspended solids concentrations for each system is illustrated in Figure 6. At present there is no specific Federal Secondary Treatment Standard for effluent suspended solids concentrations in aerated lagoon effluents.

In general, the effluent suspended solids concentration from three of the aerated lagoon systems tend to increase significantly during the warm summer months. However, two of the aerated lagoon systems (Windber, Pennsylvania and Gulfport, Mississippi) produce a relatively constant effluent suspended solids concentration throughout the entire year.

Average monthly effluent suspended solids concentrations ranged from 2 mg/l at Windber, Pennsylvania, in November, 1975, to 96 mg/l at Bixby, Oklahoma, in

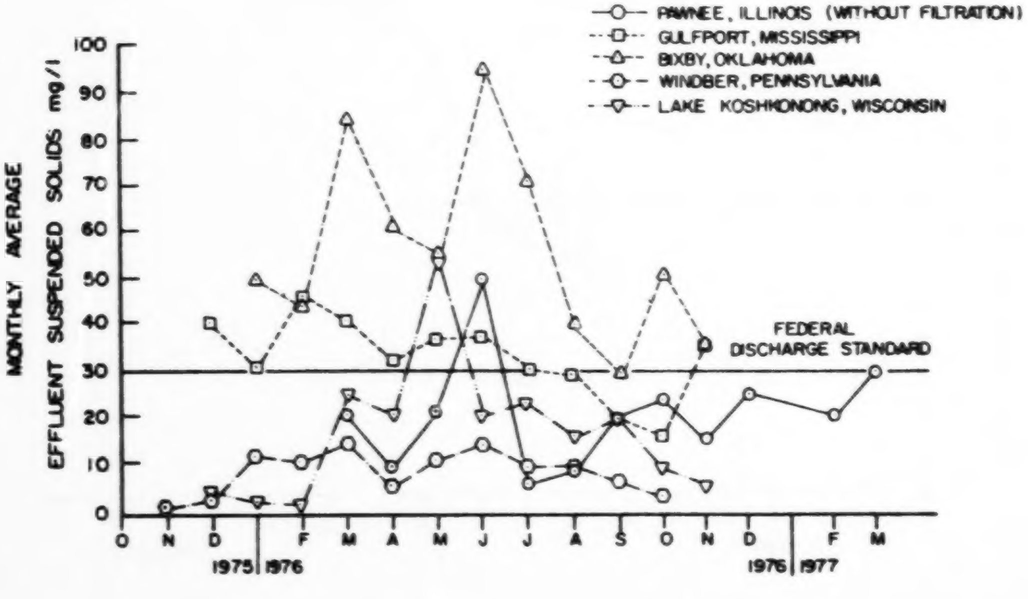

Figure 6. Aerated lagoon suspended solids removal performance.

June, 1976. The average monthly effluent suspended solids concentration for the Windber, Pennsylvania site never exceeded 30 mg/l throughout the entire study period. In addition, the average monthly effluent suspended solid concentrations of the Pawnee, Illinois, and the Lake Koshkonong, Wisconsin sites only exceeded 30 mg/l during one of the months reported.

The results of these studies indicate that aerated lagoon effluent suspended solids concentrations are variable. However, a well designed, operated, and maintained aerated lagoon can produce final effluents with a low suspended solids concentrations.

Fecal Coliform Removal
Performance

The monthly geometric mean fecal coliform removal performance of three of the five previous described aerated lagoon systems is reported in Table 1. Fecal coliform data were not available for the Lake Koshkonong, Wisconsin, site and only one monthly geometric mean fecal coliform value was available for Bixby, Oklahoma. The monthly geometric mean effluent fecal coliform concentration compared to a concentration of 200 organisms/100 ml is illustrated in Figure 7.

All of the aerated lagoon systems, except the Bixby, Oklahoma, sites have chlorination disinfection. Therefore, the data actually indicate the susceptibility of aerated lagoon effluent to chlorination. In general, the Windber, Pennsylvania, and the Pawnee, Illinois, systems produced final effluent monthly geometric mean fecal coliform concentrations of less than 200 organisms/100 ml. The non-chlorinated Bixby, Oklahoma site singe data point indicates a high effluent fecal coliform concentration. The Gulfport, Mississippi system produced an effluent containing more than 200 fecal coliform/100 ml most of the time, but the fecal coliforms were measured in effluent samples from a holding pond with a long detention time following the addition of the chlorine. Therefore, aftergrowth of the fecal coliform probably accounted for the high concentrations.

SUMMARY

From the limited aerated lagoon performance data currently available, it appears that (a) aerated lagoons can produce an effluent BOD_5 concen-

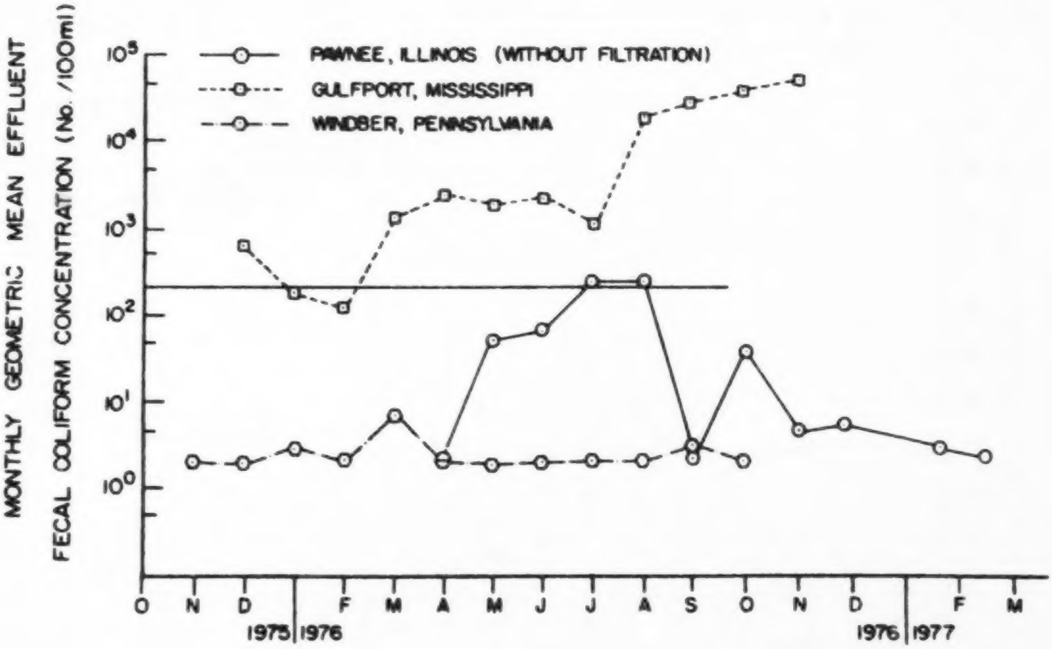

Figure 7. Aerated lagoon fecal coliform removal performance.

tration of less than 30.0 mg/l, (b) aerated lagoon suspended solids concentrations are affected by seasonal variations, and (c) aerated lagoon effluent can be satisfactorily disinfected with chlorination.

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